American Soc. of Civil Engineers

VOLUME 85 NO. HY4
APRIL 1959
PART 1

JOURNAL of the

Hydraulics Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to reader with the greatest possible speed. To this end the material herein none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to Manager of Technical Publications, ASCE. Authors should indicate the techn division to which the paper should be referred. The final date on which a cussion should reach the Society is given as a footnote with each paper. Th who are planning to submit material will expedite the review and publicat procedures by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spa on one side of 8½-in. by 11-in. paper. Papers that were originally prepared oral presentation must be rewritten into the third person before being submitt

4. The author's full name, Society membership grade, and footnote referer stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitt Because of this, it is necessary that capital letters be drawn, in black ink, ⅛ high (with all other symbols and characters in the proportions dictated standard drafting practice) and that no line of mathematics be longer than 6½ Ribbon copies of typed equations may be used but they will be proportionat smaller on the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11 paper with a 6½-in. by 10½-in. invisible frame. Small tables should be group within this frame. Specific reference and explanation should be made in the te for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11 paper within an invisible frame that measures 6½-in. by 10½-in.; the captic should also be included within the frame. Because illustrations will be reduc to 69% of the original size, the capital letters should be ⅛-in. high. Photograp should be submitted as glossy prints in a size that is less than 6½-in. by 10½ Explanations and descriptions should be made within the text for each illustratio

8. Papers should average about 12,000 words in length and should be longer than 18,000 words. As an approximation, each full page of typed te table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is co tained in the "Technical Publications Handbook" which can be obtained fro the Society.

This Journal is published monthly by the American Society of C Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michig Editorial and General Offices are at 33 West 39 Street, New York 18, New Yo $4.00 of a member's dues are applied as a subscription to this Journal. Secon class postage paid at Ann Arbor, Michigan.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

April, 1959

Papers

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

CHANNEL-SLOPE FACTOR IN FLOOD-FREQUENCY ANALYSIS

Manuel A. Benson,[1] A.M. ASCE

ABSTRACT

Annual flood peaks in New England have been related to many hydrologic :tors. The main-channel slope has been found next in importance to ainage-area size. The slope for that part of the main channel between 85 d 10 percent of the total distance above the gaging point provides the best rrelation with flood magnitudes.

Many agencies and individuals are studying the causes of floods and their ation to known meteorologic and physiographic conditions, and are develop- generalized flood-frequency relations for the purposes of hydraulic de- n, flood insurance, flood-plain occupancy, and other uses.

Data for New England have been used in the first phase of such a study by U. S. Geological Survey. The records of runoff and precipitation for this gion are as numerous and as long as records for any other region in the ited States; topographic quadrangle maps covering the entire area are ilable; historical flood data are available, dating back to the earliest es of settlement.

Peak discharge records of 170 gaging stations were used in the analysis. ese records represented all the stations in New England having at least 10 rs of record, on which the effect of regulation on annual peak discharges s known, or judged, to be negligible. These stations were located through- the region, though not as densely in the northern part. The range in the e of drainage areas was from 1.64 to 9,661 square miles. Of a total of station, 3 stations drained areas less than 10 square miles; 31 stations ined areas less than 50 square miles; 65 stations drained areas less than square miles; and 106 stations drained areas less than 200 square miles.

In practice, there are many ways of converting a set of flood data at a gle station to a flood-frequency curve or relation for that station, to

e: Discussion open until September 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1994 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 4, April, 1959.

[1] Hyd. Engr. Research, U. S. Geological Survey, Washington, D. C.

determine the floods of various recurrence intervals. It was considered tha the use of mathematically fitted curves involving prior assumptions as to distribution might obscure the very relationships that were being sought, an that graphically drawn curves, conforming as closely as possible to the original data at each station, would be most desirable. The plotting position of each flood of record was computed by using the formula $\frac{n+1}{m}$, where n is the number of years of record and m is the rank of the flood, starting with 1 as the highest. This formula has theoretical basis and also agrees closely with the length of the period of record for the top flood, thus again conforming closely to the basic data.

From the frequency curves, values were selected for the 1.2-, 2-, 5-, 10 and 25-year floods at 170 stations, and for the 50-year flood at 40 stations. Where the length of record was less than 25 years, correlation with nearby stations was used to ascertain the appropriate plotting positions within a 25-year period for the annual peaks of record. The floods for each of the state recurrence intervals were then separately correlated with a large number c physiographic and meteorologic factors. Initially, the relations were analyzed by graphical multiple-correlation techniques combined with rank-correlation methods. The following parameters were studied:

1. Drainage area
2. Slope
   - Channel slope No. 1
   - Channel slope No. 2
   - Average land slope
   - Tributary channel slope
3. Water-supply index
   - Mean annual runoff
   - Mean annual precipitation
   - Maximum 24-hour precipitation
   - 10-year 60-minute intensity
   - Total rainfall volume in 6 large storms
4. Shape factor
   - L^2/A, equivalent of basin length divided by basin width
   - L/A, basin length divided by drainage area
   - Σal, the summation of small subdivisions of the drainage area, eac multiplied by the distance of travel to the gaging point.
5. Mean elevation
6. Percentage of lakes, swamps and reservoirs
7. Stream density
8. Temperature (January average)

Most of the physical characteristics were computed as described by Langbein [1947]. "Channel slope No. 1" is one such characteristic. It is th mean slope of all parts of a channel draining at least 10 percent of the total drainage area,—thus it may include parts of the principal tributary stream "Channel slope No. 2" is the same as that described by Bigwood and Thoma [1955]; the weighted main-channel slope is averaged with the tributary channel slopes as described by these authors.

The mean annual precipitation for each drainage area was supplied by the . Weather Bureau as computed in the preparation of Hydrologic Atlas 7 [Knox and Nordensen]. Other precipitation factors were based on vari- Weather Bureau publications or on Geological Survey flood reports.

Many of the hydrologic factors, such as "channel slope No. 1" or "stream sity" were available from Water Supply Paper 968-C, Langbein, [1947], ugh not for all the 170 stations usable in New England. Because of the or involved in computing these, it was considered satisfactory, in the liminary phases of the study, to compare the various factors and test r significance using only the 85 to 95 stations for which most of the rologic factors were already evaluated.

n the graphical procedure used, Q_n, the n-year flood, was first plotted log-log paper) against the drainage area. Through the plotted points, an rage curve was drawn, and the residuals from this curve were determined each station and ranked in order of magnitude. The values for all other ameters were then ranked and the differences between ranks were used ompute the Spearman rank correlation coefficients [Wallis and Roberts, 6]. The parameter with the highest correlation (considering only one n each numbered group in the preceding table) was then used as the next iable in the graphical correlation. After the proper adjustments, a plot adjusted) first residuals versus the first parameter values led to a set of ondary residuals, which was in turn tested against the remaining parame- s.

Table 1 shows the results of graphical correlation for the 2-, 10-, and year floods for all significant parameters. This shows the decrease in standard error when successive independent variables were included. In h case, after 3 or 4 independent variables were used, further variables not appreciably decrease the standard error. It is notable that factors luating tributary slope alone were not found to be significant, once the ct of the main channel slope, as expressed in channel-slope factor no. 1, accounted for. The graphical standard errors were determined as the ge within which two-thirds of the plotted points lie. Actually these are standard errors of estimate, but are approximations adequate for these loratory studies. The standard errors are shown in terms of the original units and in the equivalent percentages these represent. One log unit als one log cycle and a decimal part of a log unit is that same decimal of the linear distance representing one log cycle.

As would be expected, the standard error increases with the increase in size of flood. It will be noted that for each size of flood, the standard or with drainage area (A) drops sharply after channel slope (S) is in- ed, and is improved by only a small amount thereafter. The average entage improvement with S is 22, 38, and 19 percent respectively, reas the other parameters after A and S show combined improvements exceeding 6 percent.

t will be noted also that water supply indices, R, P, and I, show but small ct. The relatively minor influence of rainfall is attributed to the uniform ern of precipitation throughout New England and to the unsatisfactory fall parameters so far available.

he importance of channel slope is shown by the large percentage of the r which it accounts for. However there has been no unique or universal- ccepted way of evaluating channel slope. Some hydrologists have used the drop from the head of the longest watercourse. Others have used

Table 1.--Results of Graphical Multiple Correlation

	Standard error			
	log units	percent +	percent -	percent ave
Q_2				
A	0.24	74	42	58
A, S	.155	43	30	36
A, S, R	.15	41	29	35
A, S, R, L/A	.13	35	25	30
Q_{10}				
A	.31	104	51	78
A, S	.17	48	32	40
A, S, P	.152	42	29	36
Q_{25}				
A	.26	82	45	64
A, S	.19	55	35	45
A, S, I	.17	48	32	40
A, S, I, L/A	.165	47	31	39

A=Drainage area; S=Channel slope No. 1; R=Mean annual runoff; P=Mean annual precipitation; I=10-year, 60-minute maximum precipitation; L/A=Shape factor.

weighting methods which evaluate the slope all along the main channel. Sor have used parts of the main channel, such as the lower three-quarters. Sti others have in some manner combined the slopes of tributary streams with the main channel. Part of the difficulty is that methods of defining the mai channel or of differentiating the tributary streams are not fixed.

Channel slope No. 1 was found to be the best slope factor of those evalu-ated. Its computation involves the determination of the points on the large streams at which 10 percent of the total area is drained. This is a laborio task, particularly in a regional study where it may have to be done for hundreds of stations. It was therefore considered desirable to attempt to f a simpler, and possibly better channel slope factor. It was also considere desirable to separate the effects of tributary streams from those of the ma stream. It was decided (1) to use only the main channel in a channel-slope parameter, leaving the tributary slopes for separate consideration, (2) to c fine the main channel, above each stream junction, as that channel draining the largest area, (3) to make an exhaustive study to find a way of expressi the main-channel slope that is most closely related to peak discharge.

Topographic maps were used to develop channel profiles for all 170 sta-tions in New England. At the upstream end each profile was extended to th drainage divide beyond the end of the stream shown on the topographic ma Distances were measured from the gaging point to each contour crossing (cept where these were too dense) and channel profiles were plotted from these figures.

: was considered that the most upstream part of the stream, in the steep lwaters, might affect the slope out of proportion to the volume of water ished by the headwater area. At the downstream end the slope might not ndicative of that affecting the size of peak discharges because gaging s are usually located at riffles or natural controls. It was therefore ulated that the part of the main channel whose slope would best corre- with peak discharge would be the one excluding the extreme headwater ch, and possibly some of the extreme downstream end.

The total lengths of the streams were divided into tenths below 0.7 of the ance from the gage and half-tenths beyond that. The object was to com- slopes for all possible portions of the channel, for example, 0.95 to 0.0, to 0.1, 0.6 to 0.4, etc., the figures representing in each case the fraction ne total distance upstream from the gage. There were 91 such combina- s. Two additional slope factors were computed. These were the con- ts in the regression equation relating the logarithm of the rise to the rithm of the distance from the gage, equivalent to drawing a straight line ugh the channel profile plotted to a logarithmic scale. These two factors resent an integrated slope for the entire main channel, rather than mere- ne slope between two points.

To determine the optimum slope factor, multiple correlation analyses e to be made with the 1.2-, 2.33-, 5-, 10-, 25-, and 50-year floods as de- dent variables and drainage area and each of the 93 slope factors as the pendent variables. The change from the 2-year to the 2.33-year flood made at this time in order to use a value corresponding to the mean an- flood in common use; results are comparable. The best slope factor ld be defined at the point of minimum standard error. The correlation yses were such a prodigious task that the data were prepared for auto- ic solution by a digital computer, the Datatron 205. The programming ered computation of all the slopes, the logarithmic slope factors, the dard errors with drainage areas alone, and the standard errors and cor- tion coefficients with drainage and each of the 93 slope factors. Data for stations was used in these correlation studies.

Results of the Datatron solutions are shown in Figures 1 and 2. These res show contours representing equal values of the standard error. On ure 1, for example, the standard error using the slope between points 0.5 0.4 of the total distance above the gage is 0.193 log units. Figure 1 shows he standard errors on which the contours are based whereas Figure 2 vs only the contours. Note the similar patterns shown by the controus each size of flood and the minimum in the upper left of each figure. A ghting process reveals that the slope between points 85 and 10 percent e the gage would satisfactorily give the minimum standard error for all ds, with little accuracy lost for any particular size of flood. Standard ors using the two logarithmic slope factors were higher for all floods those using the 85 to 10 percent slope.

Table 2 shows the standard errors (in log units and average percent) for size of flood, using drainage area and the "85-10" percent slope as in- endent variables. The improvement in each can be seen when slope is in- uced. A regularity shows up in the percentage of the error explained by e, which is close to 35 percent in each case. The standard error for the ear flood seems inconsistently small and the contour pattern is some- erratic. The irregularities may rise because the 50-year flood is based

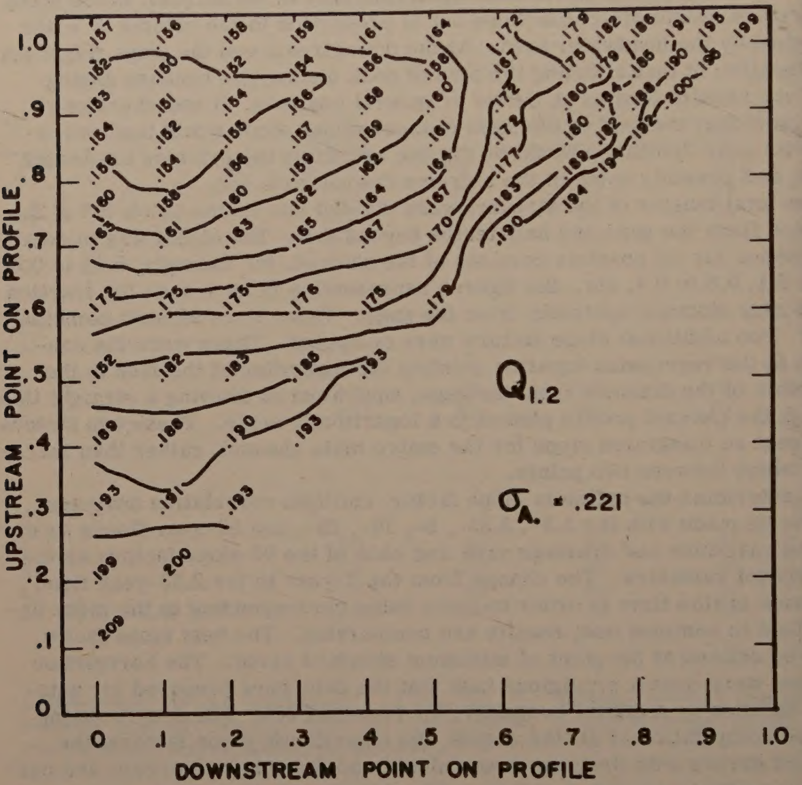


FIG. I VARIATION OF STANDARD ERROR WITH SLOPE FACTO

on 40 items, whereas the others are based on 170 items. They may also be due to the less dependable data available for the 50-year floods.

Comparison is shown with the graphical study (using channel slope No. 1 at the same stages. Results by the datatron and graphical method are not strictly comparable, first because of the difference in methods, second because 85 to 95 items were used in the graphical analysis and 170 items in t datatron analysis. However, the final datatron results, using the "85-10" slope, indicate practically the same standard error as the channel slope No. 1 used in the graphical analysis. This is gratifying because the "85-10 slope is far simpler to compute.

Figure 3 shows the typical regression equation with drainage area and slope and a plot of the regression coefficients. The consistent variation in regression coefficients is evident. (The coefficient, b, for the drainage are might well be 1.00 throughout). The multiple correlation coefficients for each of the regressions range between 0.93 and 0.97.

Opportunity still exists for improvement in the standard errors shown i Table 2, by the use of other parameters employed in the graphical study or by the improvement of parameters such as precipitation indices. Further

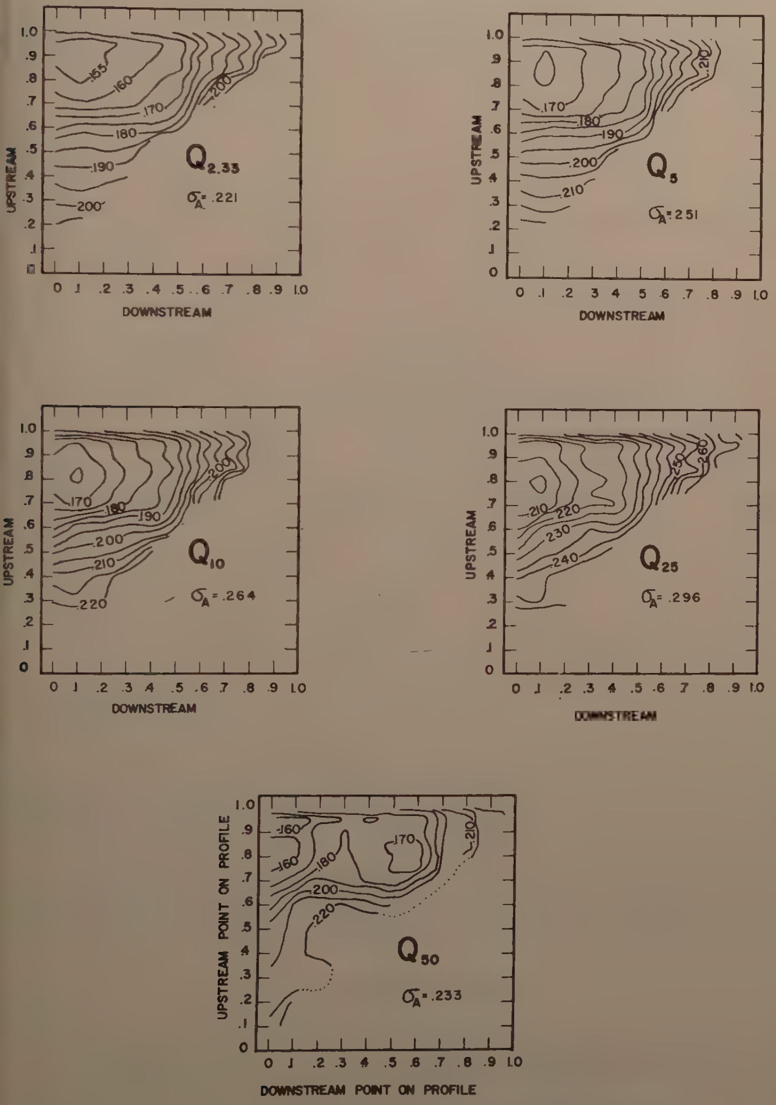


FIGURE 2: VARIATION OF STANDARD ERROR WITH SLOPE FACTOR

Table 2.- Multiple Correlation with Drainage Area and Slope (85 to 10%)

	Standard error			
	Datatron		Graphical	% Error explained by slope
$Q_{1.2}$	log units	ave. %	ave. %	
A	0.221	53	-	34
A, S	.152	35	-	
$Q_{2.33}$				
A	.221	53	58	32
A, S	.153	36	*36	
Q_5				
A	.251	61	-	38
A, S	.164	38	-	
Q_{10}				
A	.264	65	78	40
A, S	.165	39	*40	
Q_{25}				
A	.296	74	64	34
A, S	.207	49	*45	
Q_{50}				
A	.233	56	-	34
A, S	.158	37	-	
			Ave.	35

*Graphical standard error computed using channel slope No. 1.

work is proceeding along these lines. This report is presented as a progress report with a definite finding on one phase of flood-frequency analysis, and is not intended to provide the final answer.

The "85-10" percent main-channel slope found so effective for New England may not prove to be the best for other regions but it should serve as a guide in similar studies made elsewhere.

BIBLIOGRAPHY

Langbein, W. B. Topographic characteristics of drainage basins, U. S. Geological Survey, Water-Supply Paper 968-C, 155 pp., 1947.

Bigwood, B. L., and Thomas, M. P., A flood-flow formula for Connecticut, U. S. Geological Survey, Circular 365, 16 pp., 1955.

Knox, C. E., and Nordensen, T. J., Average annual runoff and precipitation in the New England-New York area, U. S. geological Survey, Hydrologic Investigations Atlas HA 7.

'allis, W. A., and Roberts, H. V., Statistics, a new approach, The Free Press, Glencoe, Illinois, 603, 604, 1956.

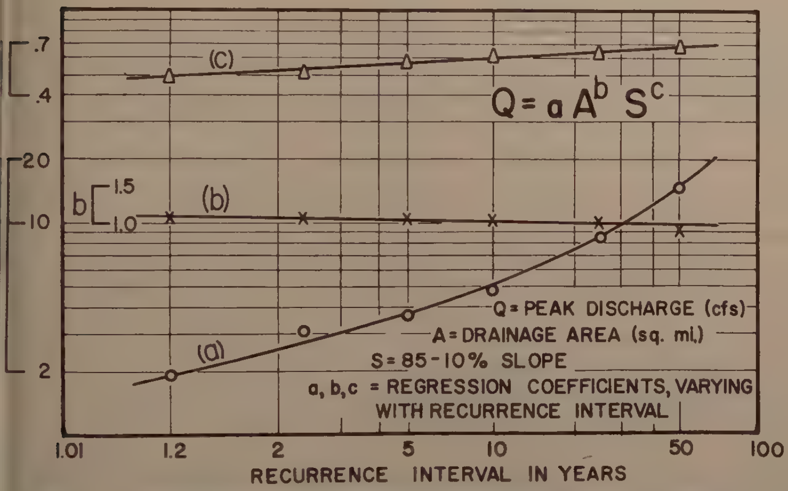


FIG. 3 VARIATION OF REGRESSION COEFFICIENTS WITH SIZE OF FLOOD

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

STORM WATER DRAINAGE IN THE CHICAGO AREA

Horace P. Ramey,[1] M. ASCE

SYNOPSIS

This paper, read before the Hydraulic Division of the American Society of ivil Engineers on February 25, 1958, is a discussion of past and recent ooding conditions in the Chicago area and some recommendations for their nprovement.

The opinions and recommendations stated herein are based on fifty years f investigation and experience with these problems. The recommendations iscussed are not meant to be specific, but rather to assist future students of ıe subject in arriving at definite solutions.

Floods have plagued mankind since the great Deluge of 2349 B.C. That was ɔout 4000 years ago. One might think that the Chicago area, much of it on a ontinental Divide, with drainage both into Lake Michigan and into the ississippi Valley, would be comparatively free from floods, but such is not e case.

To protect Lake Michigan, the source of its public water supply, from con-mination by sewage, Chicago decided many years ago to keep its drainage ıt of the lake. This eliminated the quickest outlet for storm drainage, and, ecause of the topography, placed the main outlet for such drainage at the uthwesterly corner of the Chicago area. This, and the generally flat terrain the region, has produced many problems, some of which are yet to be lved.

In the matter of flood runoff, the Chicago region is served by three reams: the Chicago River, which drains the northern area of the city itself far south as 87th Street; the Des Plaines River, which drains the westerly eas; and the Calumet River, which drains the entire Calumet region south 87th Street.

ɔte: Discussion open until September 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1995 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 4, April, 1959.

Chf. Engr., The Metropolitan Sanitary Dist. of Greater Chicago, Chicago, Ill.

A knowledge of the topography of the Chicago region is essential to the study of its flooding; and a study of the past furnishes the best data for the solution of future problems.

Topography, Drainage Areas

The most important stream is, of course, the Chicago River, which extend due west from Lake Michigan, a little more than a mile, to the Forks. From this point the North Branch extends north by northwest, about twenty-nine miles, to its source in the Skokie Valley, west of Highland Park and Lake Forest, in Lake County, Illinois. The lower six-mile reach of the North Branch of the Chicago River has been improved for navigation and flood flow, to Lawrence Avenue. Beyond that point the North Shore Channel of The Metro politan Sanitary District of Greater Chicago extends eight miles to a connection with Lake Michigan, at Wilmette. This channel (dry weather flow capacity, 1000 cfs) passes the drainage of the North Shore towns, Evanston to Glencoe, into the North Branch.

About three blocks north of Lawrence Avenue a dam was constructed to hold the North Branch of the Chicago River in its natural state, hydraulically. Above this dam, the North Branch extends westerly and then northerly about eight miles and then divides into three small streams, the West Fork, the North Branch, and Skokie River. The Skokie Marsh covered a five-mile reac of this latter river, west of Winnetka and Glencoe, and, in its natural state, served to retain certain flood water. Much of this marsh has been drained and the area used for residential building; and lagoons have been constructed to control to some extent the flow of water in the Skokie River.

The South Branch of the Chicago River, extended from the Forks (Lake Street) due south about two miles, and then southwesterly two miles further, to Ashland Avenue and 26th Street, where it divided into the South Fork and the West Fork. The South Fork extended south one and one-half miles to 39th Street and Racine Avenue. The West Fork extended west about two and one-half miles to Kedzie Avenue and 32nd Street, the easterly limit of a slough known as Mud Lake, which was the source of the South Branch. Mud Lake ex tended southwesterly, about six miles, to Harlem Avenue and 49th Street, where it connected with the Des Plaines River.

The Main Drainage Canal was constructed from the West Fork of the South Branch of the Chicago River, at Damen Avenue (one-half mile west of Ashlan Avenue) to extend southwesterly across the Continental Divide just south of Mud Lake, past Summit, and down the Des Plaines Valley, to connect with the Des Plaines River at Lockport, Illinois.

Present drainage areas of the Chicago River (above Summit) are 309 squa miles.

The Des Plaines River, rising in southeastern Wisconsin, west of Racine, flows southward, parallel to and about ten miles west of the west shore of Lake Michigan, to about 49th Street and Harlem Avenue, Chicago (one mile south of Riverside), and then makes a sharp bend to the southwest and continues past Lockport and Joliet, to Dresden, where it unites with the Kankake River to form the Illinois River. Drainage area of the Des Plaines River, above Riverside, is 643 square miles; above Lemont, 690 square miles; and above Lockport, 713 square miles.

The most important tributary of the Des Plaines River is Salt Creek, which omes in from the west at about 39th Street. Salt Creek rises in Cook County, ortheast of Palatine, flows southward through the eastern section of DuPage ounty, past Bensenville, Villa Park, and Elmhurst, to Hinsdale; and then ows eastward past Western Springs and LaGrange Park, through Bookfield, nto the Des Plaines River. Its drainage area is about 152 square miles.

Flag Creek rises in the southern part of Hinsdale and Western Springs and ows south to about 87th Street, then southwesterly and into the Des Plaines iver about half-way between Willow Springs and Sag. Its drainage area is bout 20 square miles.

The Little Calumet River rises in Laporte, Indiana, and flows westward nto Illinois at about 170th Street, then northwesterly to a point in Blue Island bout one-half mile west of Ashland Avenue, at 136th Street. Then it bends harply to the east and flows about seven miles to a junction with the Grand alumet River, at Torrence Avenue and 139th Street. From this junction the alumet River (drainage area 38 square miles) extends north about six miles, ast Lake Calumet, to Lake Michigan at 90th Street. From this same junction, he Grand Calumet River (drainage area 50.1 sq. mi.) extends eastward to the ake Michigan shore near the east line of Gary, Ind. Only a sand bar separates this river from the lake. The Grand Calumet is really a bayou, having nce been the outlet channel of the Little Calumet.

The Indiana Harbor Ship Canal connects the Grand Calumet River with Lake ichigan, through East Chicago, Indiana. The drainage area of the Grand alumet, east of this canal, is 30.8 square miles; and west of it, 19.3 square iles.

The original watershed of the Little Calumet River, above Riverdale, was bout 670 square miles. This included about 75 square miles in the eastern alf of the Sag Valley, drained by Stony Creek, a tributary of the Little alumet, and seven square miles in the Blue Island area. It also included bout 370 square miles, in Indiana, in the upper reaches of the Little Calumet, hich, since 1925 have drained directly into Lake Michigan, near Dune Park, ndiana, through Burns Ditch. Deducting these areas, total 452 square miles, rom the original 670 square miles, leaves the present watershed of the Little alumet, above Blue Island (its junction with the Calumet-Sag Channel) as pproximately 218 square miles.

The drainage area of the Little Calumet River, between Blue Island and its unction with the Grand Calumet, is eight square miles, plus thirteen square iles tributary to the Calumet storm water pumping station; total thirty-one quare miles.

The present drainage area of the Little Calumet River, as reversed, to ow into the Calumet-Sag Channel, at Blue Island, is 306 square miles.

The drainage area of the Calumet-Sag Channel, east of the Continental ivide, is about eighty-two square miles; and west of this Divide, about twelve quare miles.

The most important tributary of the Little Calumet River is Thorn Creek rainage area 115 square miles), which rises southwest of Chicago Heights nd flows into the Little Calumet in the western part of Lansing, Ill. One of e principal tributaries of Thorn Creek is Butterfield Creek (drainage area bout thirty square miles), which rises in the southwest corner of Cook ounty, west of Matteson, and flows through Olympia Fields, and then south of lossmoor and Homewood, to join Thorn Creek in Glenwood. An easterly ibutary of Thorn Creek is Deer Creek (drainage area about sixteen square

miles), which rises in Crete and flows northerly through East Chicago Heights, and then northwest along the Glenwood-Dyer Road, into Thorn Creek in Glenwood.

The area south of 159th Street and west of the Illinois Central Railroad is drained by the Calumet Union Drainage Ditch (drainage area about twenty-fou square miles), which runs easterly from Kedzie Avenue, along the line of 163rd Street, and into the Little Calumet River in South Holland.

Summary—General Topographic Characteristics

The area discussed herein may generally be described as flat and rolling and although there are many ridges and divides in the area, they are not obviously distinct.

The existence of a large flood plain is evident in those areas containing ponds and sloughs as well as the areas bordering many of the shallow stream beds.

Most of the streams in the area flow parallel to the lake shore as a result of the terminal morraines created by the receding glaciers.

Changes in Stream Conditions

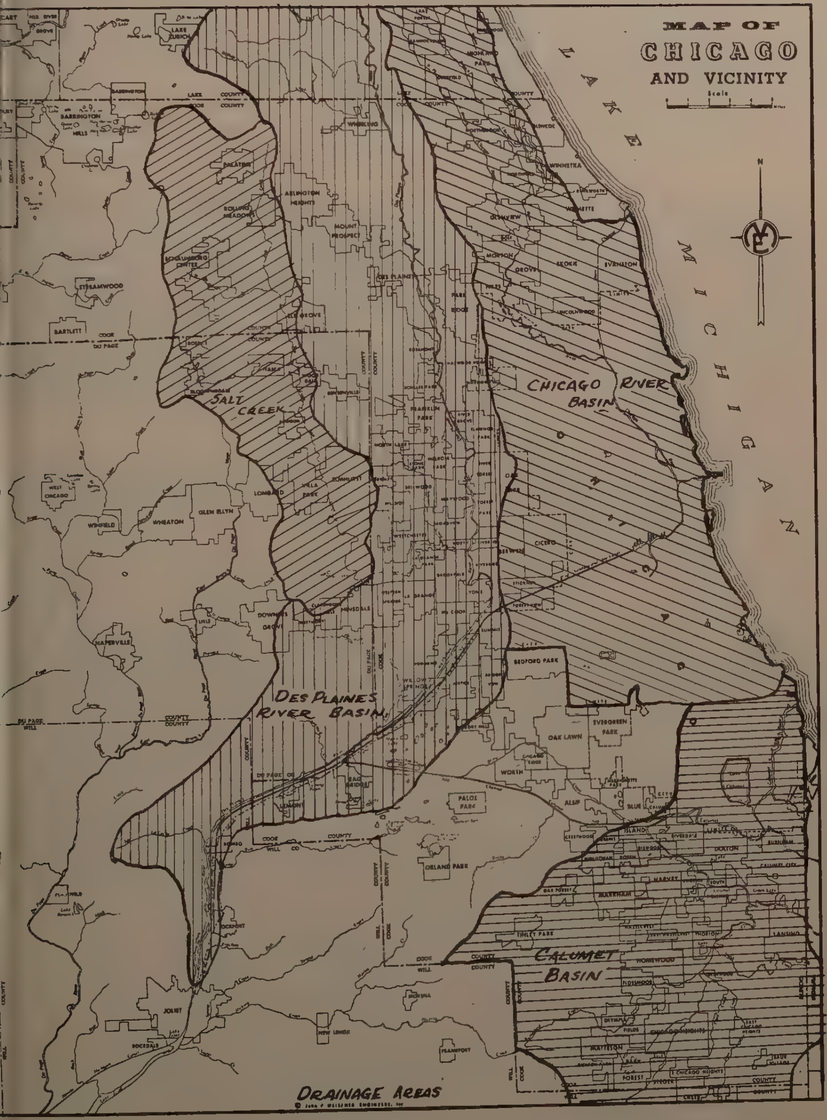
The first change from the natural conditions of these streams, in regard to storm flow, occurred in 1848, when the Illinois and Michigan Canal was opened, from the South Branch of the Chicago River at Bridgeport (Ashland Avenue) through the Des Plaines Valley, to Lockport. About a year later, the Calumet Feeder of the I. & M. Canal went into service, from the Calumet River near Blue Island, through the Calumet-Sag Valley to the I. & M. Canal, at Sag.

Between 1867 and 1871, the Illinois and Michigan Canal was deepened, fro Bridgeport to Lockport, for a gravity flow of 1,000 cfs from the Chicago River, in lieu of about 250 cfs previously pumped.

In 1871, the Ogden-Wentworth Ditch was dug, from the Des Plaines River, through Mud Lake, to the West Fork of the South Branch of the Chicago River The purpose was to drain Mud Lake, to enhance local real estate values. Thi ditch was 20 feet wide and its bottom was below the bed of the Des Plaines River. In the spring of 1872, the Des Plaines River floods greatly enlarged the new ditch, depositing the soil in the I. & M. Canal.

Thereafter, more flood water than normal came from the Des Plaines River into the West Fork of the South Branch. Even the dry weather flow of the Des Plaines River followed the same course and then flowed down the I. & M. Canal. This condition continued until June 1877, when the Ogden Dam was built.

The Ogden Dam was a row of sheet piling, 50 feet long, with crest at elevation +11.8 CCD, which was about two feet lower than the natural banks of the Des Plaines River, and a foot or two higher than the surface of Mud Lake It was built in the arm of the Des Plaines River which led to Mud Lake, west of Harlem Avenue, near 49th Street. This dam was rebuilt in 1885, of stone-filled timber cribs, with its crest at the same elevation. The new dam serve until 1894, when the Riverside Spillway, 397 feet long, at elevation +16.25 CCD, was built on the east side of the Des Plaines, a short distance north of Ogden Dam.


MAP OF
CHICAGO
AND VICINITY
LAKE
MICHIGAN
CHICAGO RIVER BASIN
SALT CREEK
DES PLAINES RIVER BASIN
CALUMET BASIN
DRAINAGE AREAS

The Riverside Spillway was closed in 1908 when the Willow Springs Spil way was built, to replace it, and discharge a like amount of flood water fro the Des Plaines River into the Drainage Canal, instead of into the South Branch of the Chicago River. The Willow Springs Spillway was closed in 1955, and since that date Des Plaines River floods have not affected the flo in either the Chicago River or the Drainage Canal.

Chicago Drainage Canal

The greatest change in stream conditions in the Chicago region was the construction of the Chicago Drainage Canal, 1892-1900, from the West For of the South Branch of the Chicago River, at Damen Avenue, twenty-nine miles, to the Des Plaines River at Lockport. This was later extended, 190 1907, four miles, to connect with the Upper Basin of the Illinois and Michi Canal and the Des Plaines River, in the northern part of Joliet. This chan was designed, 1892, with a normal hydraulic capacity to flow 10,000 cfs of water, to carry the runoff of the maximum storm expected in the Chicago for the next thirty years. Its capacity, in the rock section, fifteen miles, f Willow Springs to Lockport, is 14,000 cfs.

Construction of the Drainage Canal was accomplished by the improvem of the South Branch of the Chicago River, to capacity to flow 10,000 cfs of water at a velocity of less than 3 feet per second.

North Shore Channel

The North Shore Channel was constructed, 1908-1910, from Lake Michi at Wilmette, southwest about two miles to Emerson Street, Evanston, and then due south on the line of Kedzie Avenue, six miles, to Lawrence Avenu It joined the North Branch of the Chicago River about three blocks north o Lawrence Avenue. Its nominal flow capacity was 1000 cfs.

Prior to the construction of the North Shore Channel, the marshy land around Emerson Street, and southward, drained south through a small bro into the Chicago River, south of Foster Avenue. The North Shore Channel added no significant drainage area to the North Branch of the Chicago Riv but it did speed the runoff of flood water.

Calumet-Sag Channel

The Calumet-Sag Channel was constructed, 1911-1922, from the Little Calumet River near Blue Island, sixteen miles through the Sag Valley, to nect with the Main Drainage Canal, at Sag, about three miles upstream fr Lemont. The nominal flow capacity of this channel was 2000 cfs. Its dept was 20 feet and its minimum width 60 feet. This channel is now being widened, by the Federal Government, for navigation, to a bottom width of feet, at a depth of 9 feet below water. This will increase its nominal hydr capacity to 5000 cfs.

Summary Regarding Man Made Changes—General Effects

The areas discussed in this paper are densely populated and highly ind alized. Some of the stream characteristics have been altered by dredging the addition of many man made structures.

n effect of these man made changes has been to change, in some cases, outlying borders of the drainage basins. This, together with the general graphic characteristics of gently sloping plains and flattish slopes, pre-s a drainage problem which requires bold and imaginative solution. onsiderable progress has been made in the past with the creation of the ting open channel system, as well as the sewer program presently under It is, however, obvious that the tremendous population explosion and d rate of urbanization require immediate action to remedy a deteriorating ation.

duction to Flood History

onsiderable material describing early floods in the Chicago region is lable in the form of historic rhetoric.

ome rainfall data as far back as 1875 is available for use. This data is, ever, not sufficiently complete to afford any scientific analysis of their ef- The following briefly summarizes the general impressions of those re-led floods and periods of intense rainfall.

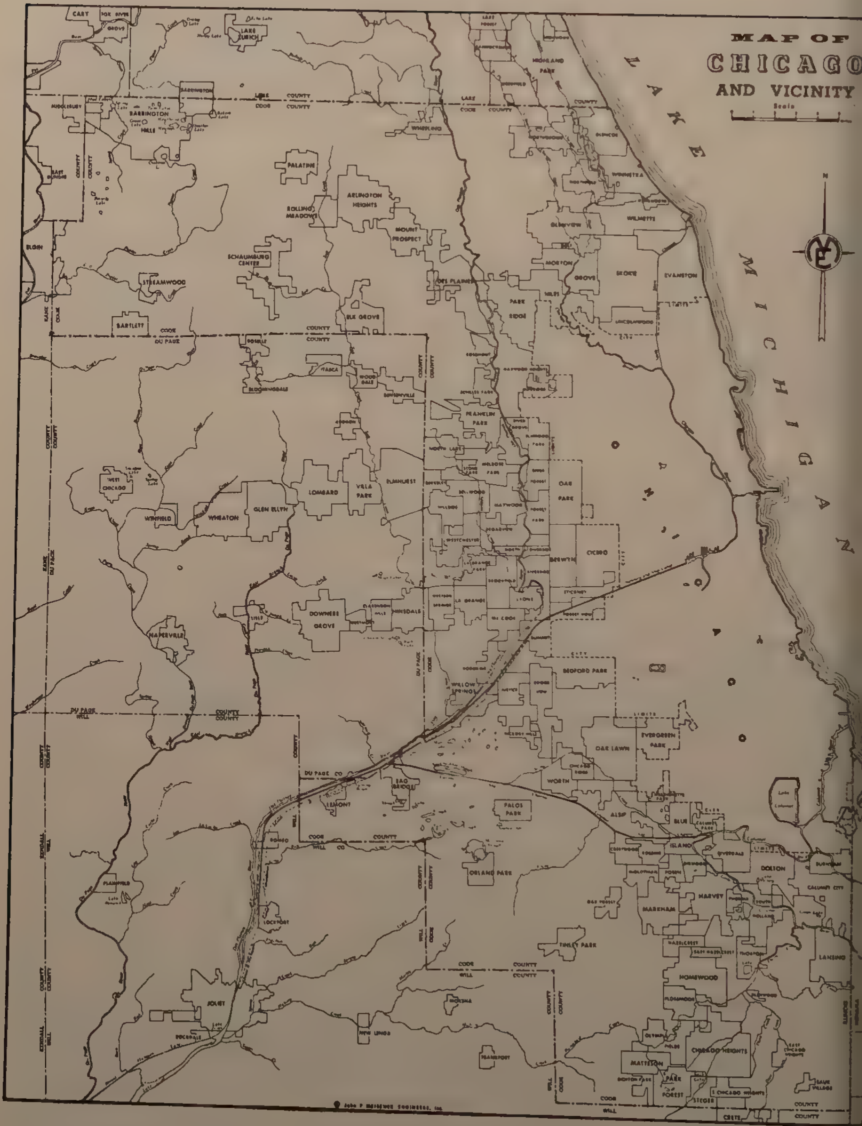
Early Floods

he first record of a flood in the Chicago region was March 29, 1674, by explorer priest, Marquette. He and two other Frenchmen and a number dians, on their way from Green Bay to the principal town of the Illinois ans, had spent the winter, after December 4, 1673, near Damen Avenue the West Fork of the South Branch of the Chicago River. On March 29, , he was driven from his camp by high water coming through Mud Lake, the Des Plaines River, with the spring break-up of the ice. He proceed-he next day, up the South Branch, through Mud Lake, without any portage, then on down the Des Plaines River, which was in flood (Brown's ainage Channel and Waterway," pages 103-104).

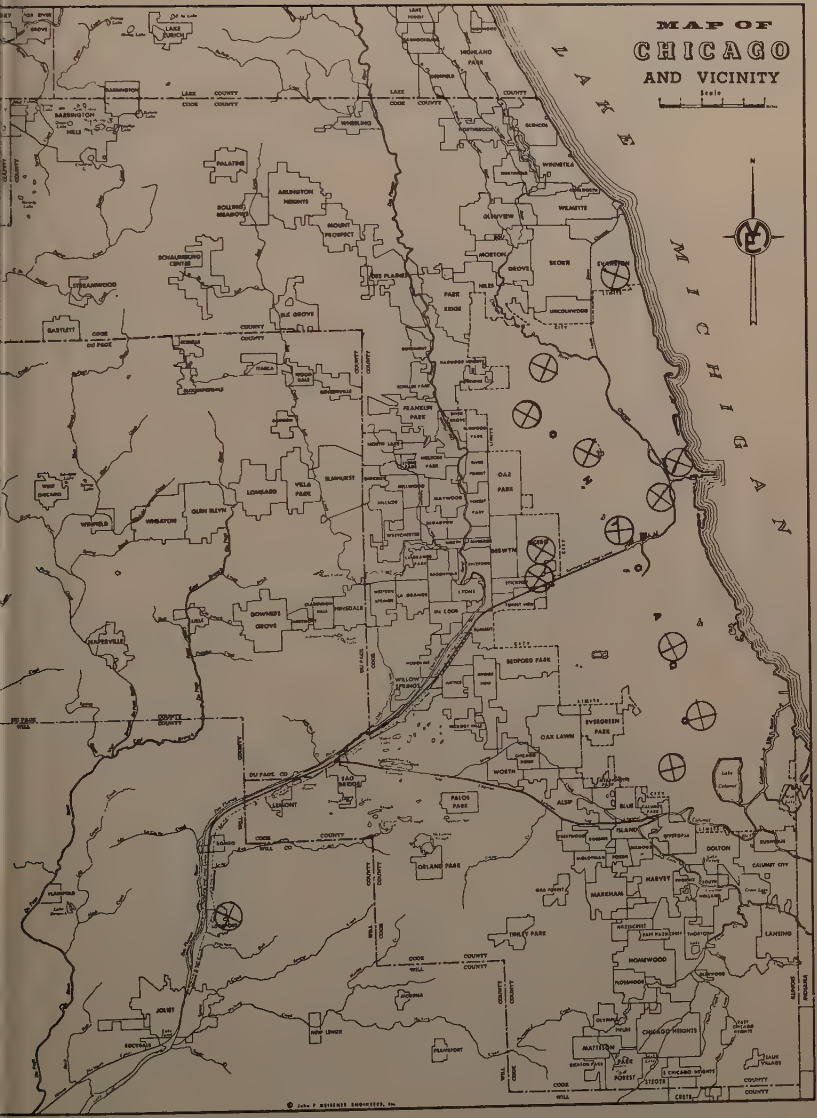
Flood of March 12, 1849

most disastrous flood occurred in the Chicago River, March 12, 1849. recipitation records are available, but this flood followed the pattern of ious floods coming in from the Des Plaines River, through Mud Lake, into South Branch. An ice jam formed in the South Branch, south (upstream) e Madison Street bridge and raised the water and floating ice two to three The jam broke at 9 A.M., March 12, and the rapid current swept away y vessel moored below that point. The mass of ice and sailing vessels ck the Madison Street bridge, wrecked it, and added the bridge to the float-lebris. Next, the Randolph Street bridge was swept away, as were a ber of canal boats and sailing vessels moored in the South Branch, north andolph Street.

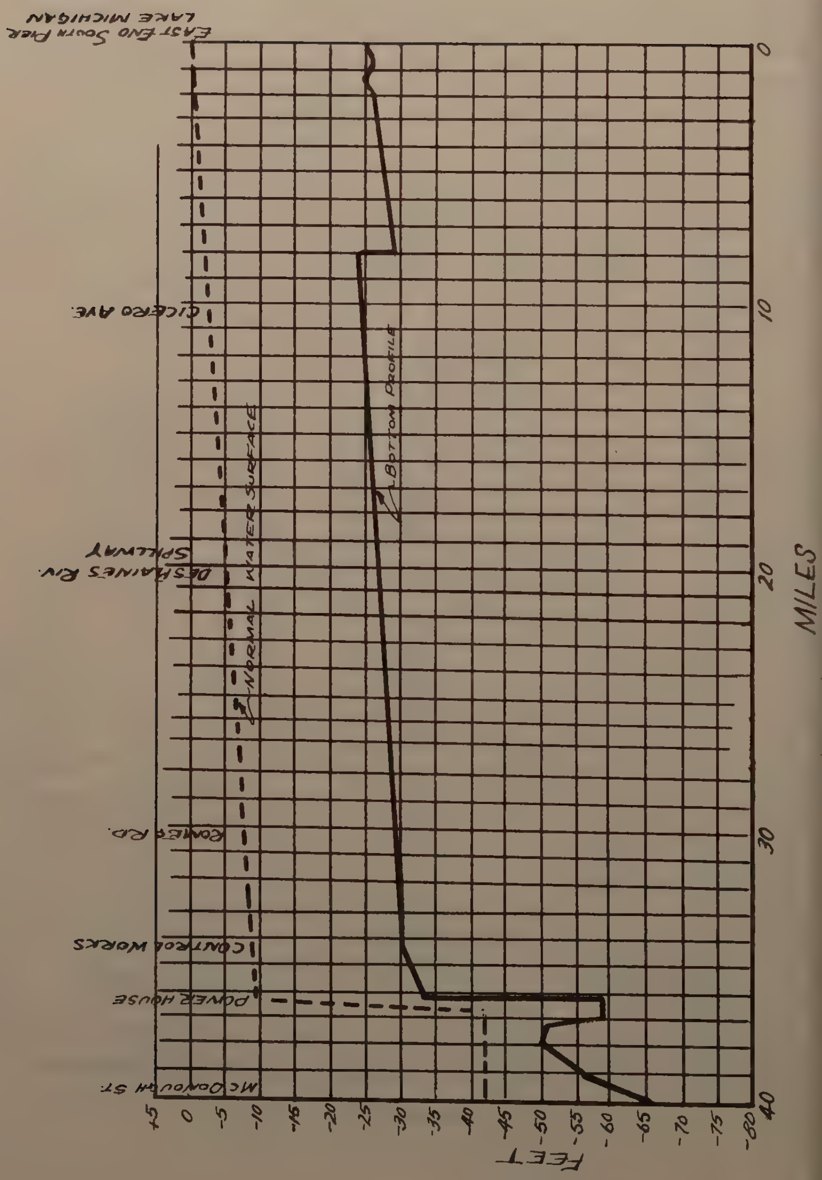
he mass of ruin then swept into the main river, where the banks were with steamers, schooners, brigs, canal boats, scows, etc. The Wells et bridge was open and the middle of the stream clear, but the floating s of vessels and wreckage extended from bank to bank and soon swept both ends of this bridge and mingled it with the general wreckage. The er vessels crashed together, entangling booms, bowsprits, and rigging.

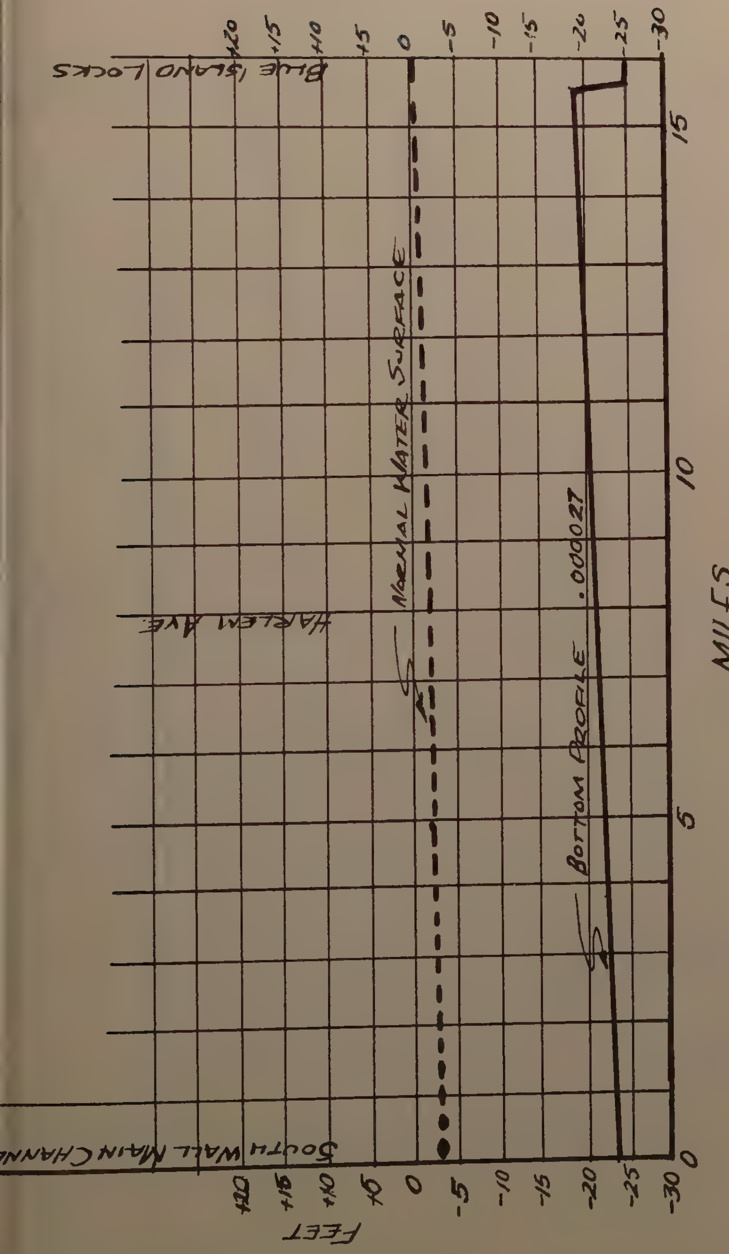


LOCATION MAP

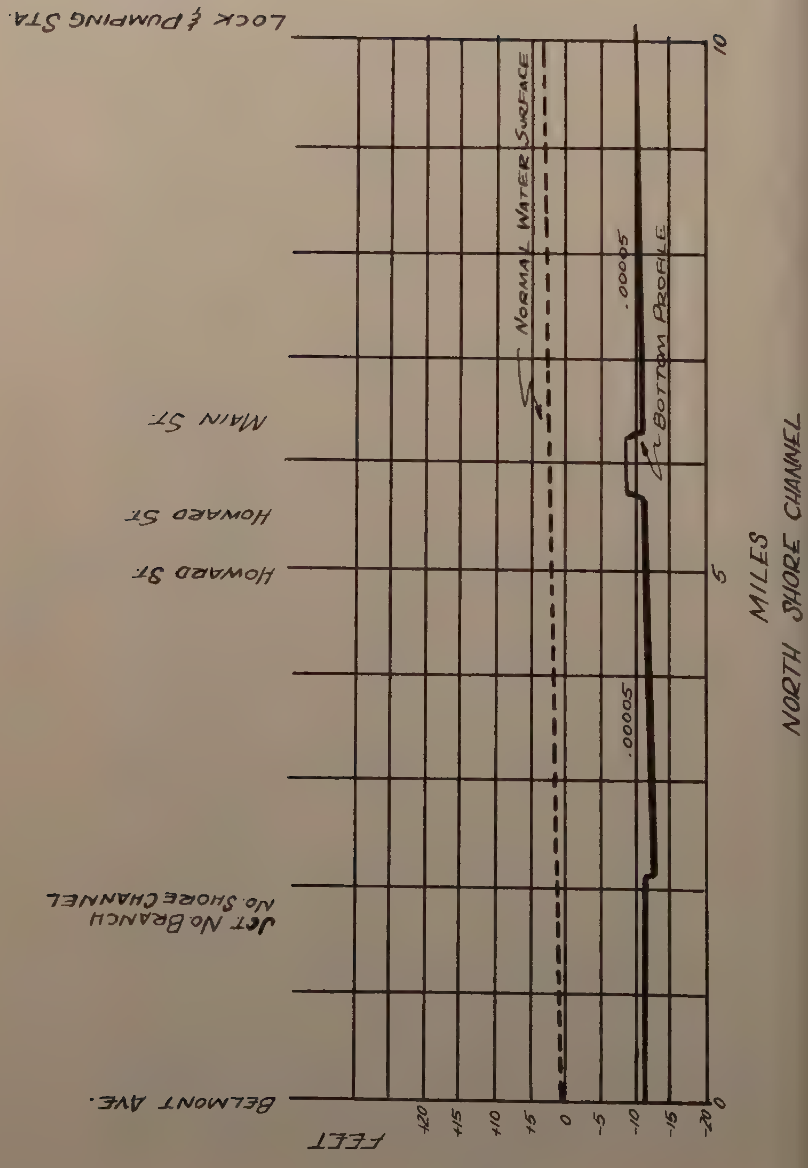


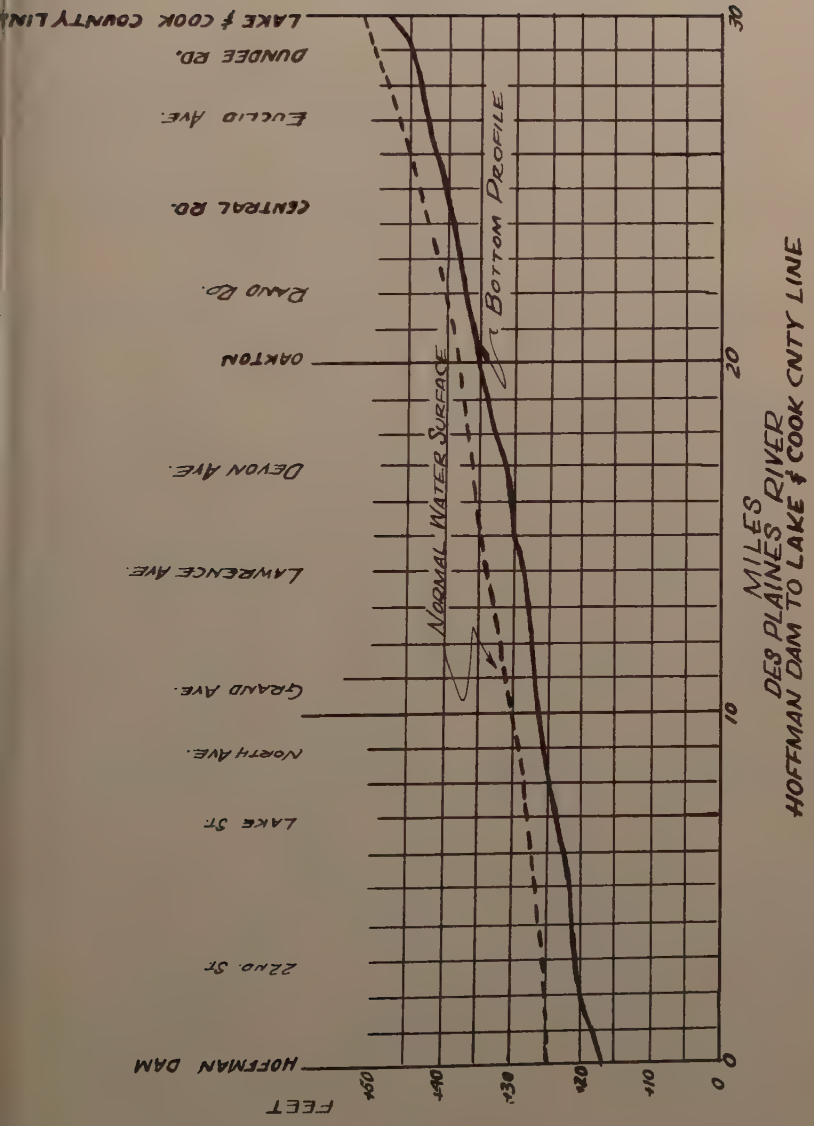
⊗ PRECIPITATION STATIONS FURNISHING DATA


EAST END SOUTH PIER
LAKE MICHIGAN
CICERO AVE.
DES PLAINES RIV.
SPILLWAY
ROMEO RD.
CONTROL WORKS
POWER HOUSE
MC DONOUGH ST.
NORMAL WATER SURFACE
BOTTOM PROFILE
MILES
FEET

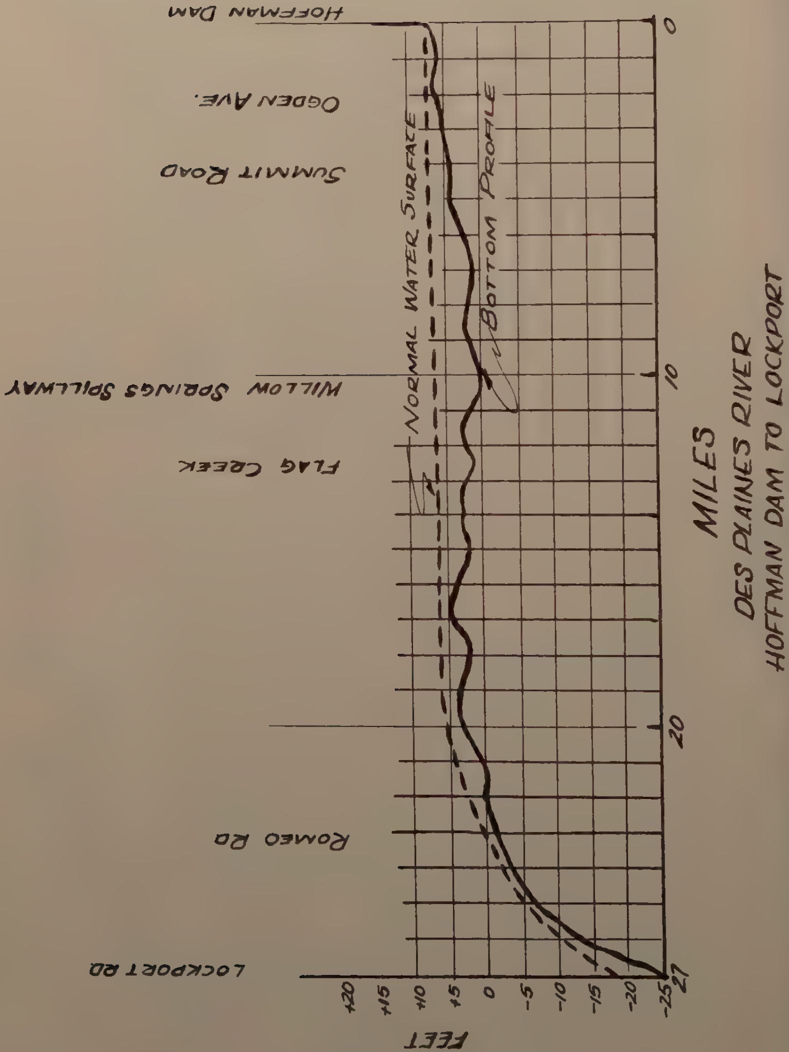


CAL SAG CHANNEL PROFILE


LOCK & PUMPING STA.
MAIN ST.
HOWARD ST.
HOWARD ST.
JCT. NO. BRANCH
NO. SHORE CHANNEL
BELMONT AVE.
FEET
+20
+15
+10
+5
0
-5
-10
-15
-20
NORMAL WATER SURFACE
BOTTOM PROFILE
.00005
.00005
0
5
10
MILES
NORTH SHORE CHANNEL



DES PLAINES RIVER
HOFFMAN DAM TO LAKE & COOK CNTY LINE
MILES
FEET
NORMAL WATER SURFACE
BOTTOM PROFILE
LAKE & COOK COUNTY LINE
DUNDEE RD.
EUCLID AVE.
CENTRAL RD.
RAND RD.
OAKTON
DEVON AVE.
LAWRENCE AVE.
GRAND AVE.
NORTH AVE.
LAKE ST.
22ND ST.
HOFFMAN DAM
0
10
20
30
+50
+40
+30
+20
+10
0


HOFFMAN DAM
OGDEN AVE.
SUMMIT ROAD
WILLOW SPRINGS SPILLWAY
FLAG CREEK
ROMEO RD
LOCKPORT RD
NORMAL WATER SURFACE
BOTTOM PROFILE
0
10
20
27
MILES
+20 +15 +10 +5 0 -5 -10 -15 -20 -25
FEET
DES PLAINES RIVER
HOFFMAN DAM TO LOCKPORT

smaller vessels were crushed. The ice, in places, was wedged several above water level.

The Clark Street bridge was completely swept away and a great many of larger vessels became wedged together a short distance east of Clark eet, and with the general wreckage formed a dam, by 12 o'clock noon. ve this dam, ice and wreckage accumulated to considerable height; and ow it, numbers of vessels were swept into Lake Michigan, many of which e wrecked.

The dam still held at 10:00 A.M. the following day, but most of the ice had sed under the vessels and the water level had fallen about two feet. The sis was over. Fears that the ice in the North Branch would break up and to the damage, were not realized.

The flood was general and caused considerable damage in the Des Plaines Illinois Valleys.

Other Floods

Other floods occurred in Chicago, due to the following extremely heavy ns:

	Rainfall
September 9, 1875	3.44 inches
July 25-26, 1878	4.14 inches
July 6-7, 1879	3.25 inches
November 11-12, 1881	3.35 inches
November 5-6, 1883	3.39 inches
March 25-26, 1884	3.26 inches
June 2-3, 1885	3.44 inches

However, the records available are not sufficient for any study of these ods.

mary of Runoff from Past Floods

For convenience in study, the principal data regarding past floods in the cago area are listed in the tabulation following:

	Rainfall Inches	Time Hours	Runoff cfs
August 2-3, 1885	6.33	27	?
May 6-8, 1892	?	?	10,000
August 11, 1908	4.34	24	9,000
April 29, 1909	2.92	36	8,900
August 14-15, 1909	3.52	24	10,000
March 17-18, 1919	2.31	48	10,600
May 3-6, 1919	2.77	72	9,000
Sept. 11-13, 1936	4.58	39	15,000
July 1, 1938	4.14	16	12,000
July 6, 1943	3.9-4.54	24	16,000

April 5-7, 1947	5.27	24	29,100
March 19, 1948	3.50	13	24,500
July 14-15, 1949	4.64	24	16,000
June 2, 1950	4.38	24	13,000
July 16-17, 1950	5.40	24	16,000
May 10-11, 1951	3.91	24	16,000
October 9-11, 1954	6.66	28	27,000
	7.20	47	
July 12-13, 1957	6.00	6	28,000
	6.34	23	

The conclusion to be drawn from a study of these floods is that in the Chicago area within the past thirty years the runoff, from one cause or another, has increased at least two and one-half times. The Chicago Drainage Canal, planned in 1892 to carry the runoff from the greatest flood which migh occur in the Chicago area, then estimated at 10,000 cfs, has now become inadequate. Many changes have taken place in the Chicago area since that tim sixty-five years ago, and the whole storm drainage problem must be reevaluated. Apparently outlet capacity of 30,000 to 35,000 cfs will be needed.

Floods in the Des Plaines River

The Des Plaines River has a drainage area of 643 square miles, above Riverside. Of this, some 152 square miles are in the watershed of its princ pal tributary, Salt Creek. The drainage area of Salt Creek, above the Fuller burg dam, just north of Hinsdale, is about 122 square miles.

From the Hoffman dam, in Riverside, the Des Plaines River extends thirt one miles north to the Cook-Lake County line, just north of Wheeling, and or fifty-seven miles further to its source in Wisconsin, west of Kenosha and Racine. Downstream from Riverside this river extends southwesterly, abou twenty-six miles, to Lockport. The slope in the flood plain of the river, and in the river itself, averages about 1.0 feet per mile between Riverside and th north line of Cook County, and about 1.9 feet per mile between Lockport and Riverside.

The average cross section of the Des Plaines River, between Riverside a North Avenue, for dry weather flow, is slightly less than 400 square feet; an the section between North Avenue and Wheeling is probably less. This mean a channel about 150 feet wide and less than 3 feet deep.

The Forest Preserve District of Cook County, in conjunction with the Sta of Illinois, has adopted a project to control the low water flow of the Des Plaines River, which at times is as low as 5 cfs, for weeks on end, in the thirty-one mile stretch between Riverside and the Cook-Lake County line. This involved the construction of twelve to thirteen low head dams, to provid at least two feet of water above each dam. Dams at Dempster Street, Devon Avenue, and Touhy Avenue were built in 1946. The Hoffman dam, in Riversi was rebuilt in 1950-1956. Dams at Hintz Road, one mile south of Wheeling, and at Foundry Road, were to replace existing dams. One other dam was bu in 1957.

At least twenty-nine highway and nine railroad bridges cross this stretch he Des Plaines River, the average horizontal clearance of such bridges ng about 150 feet. The average loss of head through these bridge openings imes of high water, may be 3 to 4 inches per bridge. The flood stage at th Avenue may therefore be 3 to 4 feet higher because of losses through se restricted openings. The flood stage at Wheeling may be increased ce as much. Accumulated drift at bridges and other narrow places has, no bt, caused further losses, for there is no record of this river having been ared during the past twenty-five years.

Some recent floods in the Des Plaines River, as measured at Riverside, as follows:

	Riverside Gauge	Maximum Discharge cfs	Runoff cfs per sq. mi.
April 21, 1881	-	13,500	21.0
February 9, 1887	20.2	10,324	16.1
June 24, 1892	21.3	13,000	20.2
March 25, 1904	21.45	10,780	16.8
March 7, 1908	21.47	10,810	16.8
May 1, 1909	21.97	11,620	18.1
March 18, 1919	22.17	7,450 ?	11.6
May 9, 1933		5,200	8.1
Sept. 17, 1936		5,160	8.1
July 3, 1938		5,350	8.3
March 16, 1944		4,600	7.2
April 6, 1947		3,780	5.9
March 20, 1948		6,510	10.1
April 26, 1950		6,340	9.9
October 11, 1954		6,340	9.9

The measurement, April 21, 1881, was made by Lyman E. Cooley, of flow r a weir. The measurement, February 9, 1887, was made by T. T. nson, using rod floats.

The flood in April 1881 was due to the melting of ice and snow, amounting bout one foot of water, which had accumulated on ground which was practi-y saturated when winter set in.

Four floods in Salt Creek, as measured at the Fullersburg dam, in 1876, 1, 1886, and 1887 produced flows of about 2800 cfs. In these cases, the off was about 22.6 cfs per square mile, from the 122 square miles in the ershed above Fullersburg. The actual measurement in 1881 was 2760 cfs.

The floods of 1881, 1892, and perhaps of 1909 may be classed as great ds. It is fortunate for the residents of the Des Plaines Valley that no at flood has occurred in the Des Plaines River since 1909. The highest er levels in this time were in March 1919; and these levels were about

3.5 feet below the levels due to the 1881 flood. The recorded discharge at Riverside, March 18, 1919, was 7450 cfs. However, with a discharge of 651[illegible] cfs March 20, 1948 and 6340 cfs April 26, 1950, the water levels in the Des Plaines River between Madison Street and North Avenue closely approached the high levels of March 1919. This indicates some obstruction to flow and the need for clearing the channel.

A report on the "Improvements of the Des Plaines River" was made July 1924, by Ernest L. Cooley, Hydraulic Engineer, to Edward J. Kelly, Chief Engineer of The Sanitary District of Chicago. For the critical reach betwee[n] the Hoffman dam, at Riverside, and North Avenue, the improvement recommended was as follows:

Lower the crest of the Hoffman dam from +25.56 to elevation +20.00 C.C.D.;

Excavate the channel from the Hoffman dam, 4800 feet to the junction with Salt Creek, to a bottom width of 300 feet and a bottom grade of +15.00;

Excavate the channel, from the Salt Creek junction to a point about a mile north of North Avenue (Elmwood Park), to a bottom width of 200 feet and a bottom grade rising to +17.00 at 22nd Street, +20.00 at Lake Street, and +22.00 at North Avenue;

Construct a sub-dam at 22nd Street with crest at elevation +22.00;

Lower the crest of the Public Service dam, below Lake Street, from +28.54 to elevation +25.00.

This plan, estimated to cost several millions of dollars, would lower the flood water stage by 5 or 6 feet, from the Riverside dam to North Avenue a[nd] beyond. The dams would cause low water boating pools, 18,000 feet long through Riverside to 22nd Street, 16,000 feet long from 22nd Street to Lake Street, and 15,000 feet long from Lake Street to Elmwood Park.

This was a good plan in 1924 and it is a good plan today, with proper mod[i]fications, including its extension to the north line of Cook County. Recent studies indicate that the bottom grade of the channel should be about +13.00 C.C.D. from the Hoffman dam to the mouth of Salt Creek and then rise, on a slope of 0.002, to about +17.20 at 12th Street, +21.20 at North Avenue, +33.0[0] at Golf Road, +36.00 at Lake-Euclid Avenue, and +45.00 at the north line of Cook County. The depth, below high water, would be 10 feet. Bottom width [of] the new channel would be 300 feet, from the Hoffman dam to the mouth of Sa[lt] Creek, then about 230 feet to North Avenue, then 210 feet to Higgins Road, diminishing to 150 feet at Palatine Road. Changes in channel widths would [be] made at points where tributary creeks enter the Des Plaines River. To ma[in]tain desired low water levels, sub-dams should be constructed, at proper intervals. To lessen the rise in water level over these dams, to pass high flood flows, the dams could be built diagonally instead of directly across th[e] stream, and thus be given greater crest lengths. Growth of shrubbery, whi[ch] would be obstructive to flood flow, should be restricted.

It is believed that channel improvements, such as those suggested above, would enable the Des Plaines River to carry any flood which may reasonabl[y] be expected in the future. The depth of cut, below the bed of the present channel, would probably average 4 to 5 feet, between the Hoffman dam and North Avenue, and perhaps 3 feet between North Avenue and the County Lin[e]. The construction cost might be \$6,000,000 to \$9,000,000.

Improvement would also be needed in Salt Creek, Addison Creek, Silver ›ek, and other tributaries. Reports have been made, by the Division of terways of the State of Illinois, indicating that such construction might ›t, as follows:

Addison and Silver Creeks	$4,790,520
Flag Creek	1,836,170
Salt Creek	5,000,000

Flag Creek (drainage area about 20 square miles), which rises in the thern part of Hinsdale and Western Springs, north of 47th Street, is a all stream, about seven miles long. The Illinois Toll Road (now under con- uction) parallels Flag Creek, from 71st Street, north to about 47th Street, l crosses it. Considerable flooding has occurred, in the past, in Hinsdale l Western Springs, because of the inadequate natural channel of Flag Creek.

In recent years a considerable amount of building has been permitted on lands subject to overflow from high floods in the Des Plaines River. Inno- it purchasers of houses in such areas have suffered unexpected and un- rranted damage from high water. No public agency should approve such astruction, regardless of the pressure applied by subdividers and builders.

tle Calumet River

A study of floods in the Little Calumet River indicates that storm runoff es in the Calumet area have at least doubled in the past fifty years. The inage area of this stream has been subject to several changes, over the ırs. The original 670 square miles of the Little Calumet above Riverdale, luded seventy-five square miles in the Sag Valley and seven square miles ound Blue Island, now tributary to the Calumet-Sag Channel, as well as 370 ıare miles in Indiana which now drain to Lake Michigan through Burns ch.

Records of the worst floods are as follows:

	Runoff	Drainage Area Sq. Mi.	Rate per Sq. Mi.
Riverdale			
March 6-7, 1908	11,000 cfs	670	16.4 cfs
North Harvey			
June 11, 1939	3,440 cfs	198	17.4 cfs
March 16, 1944	4,305 cfs	198	21.7 cfs
Blue Island			
April 5-7, 1947	9,100 cfs	306	29.8 cfs
March 19, 1948	5,000 cfs	306	16.3 cfs
October 9-11, 1954	6,700 cfs	306	21.9 cfs
July 12-13, 1957	7,200 cfs	306	23.5 cfs

A new lock and control gates are to be built in the Calumet River as part of project for widening the Calumet-Sag Channel, now under construction. s will leave only the thirty-eight square miles of the Calumet River water- d tributary to Lake Michigan. The remaining 268 miles of the present 306

square miles of watershed of the Little Calumet, Grand Calumet, and Calum
will be tributary to the Calumet-Sag Canal.

For present flood relief into Lake Michigan, a channel 3,500 to 4,000 square feet in cross-section is available. In the future, only the capacity of the control gates at the new lock will be available.

It would appear that the minimum capacity of the Calumet-Sag Channel, fo the passage of the runoff from a great flood, should be about 10,000 cfs. Thi much, or more, could come from a runoff of 30 cfs per square mile, from th 306 square mile watershed east of Blue Island and the 96 square miles in th Calumet-Sag valley. About 20 per cent of the 221 square miles within the city limits of Chicago, or 44 square miles, south of 87th Street, are includec in the area draining to the Little Calumet River and the Calumet-Sag Channe

The channel of the Little Calumet River, upstream from Blue Island, is quite inadequate for storm water flow. In 1941 the State of Illinois improvec this channel for a 50-foot width, from the Indiana State line to the mouth of Thorn Creek; and in 1946 extended this 50-foot improvement to Roll Street, in Blue Island. Under present conditions of greatly increased runoff due to the considerable building in the area, it appears that the width of this channe should be more than doubled and perhaps the depth should be increased. Op ings at the many bridges should be greatly increased. A pending improveme in Thorn Creek, already authorized by the State of Illinois, will increase the rate of storm water flow; and direct attention to the need for further improvement in the Little Calumet River above Blue Island.

Summary—Floods: Need for Remedial Measures Obvious

The foregoing has outlined chronologically, the history of storms and resultant floods in the Chicago region. As can be seen, there exists some dat of recent years which can be used to appraise and investigate a few past storms and their relation to flooding. It becomes more obvious upon such e amination that immediate remedies must be found for the flooding situation the area. The flood frequency is becoming greater with successively small rainfall and promises to become still greater with further urbanization and growth.

City of Chicago

The City of Chicago embarked on a program of construction of storm rel sewers, in 1947, expected to entail an ultimate expenditure of $165,000,000, to catch up with existing needs. Three bond issues to provide funds for this construction have already been approved by the voters, as follows:

1946	$58,160,000
1955	30,000,000
1957	8,000,000
Total approved	$96,160,000
Future	70,000,000
Total	$166,160,000

. is hoped that this work can be carried on at the rate of about $15,000,000 year, 1957 and after.

n addition to the work by the City of Chicago, The Sanitary District of cago expended $47,367,000 on its system of South Side relief sewers (to ember 31, 1956); and is currently spending about $5,200,000 on an extion of the Racine Avenue Pumping Station, for pumping storm water only. total expenditure, by the Sanitary District, for this storm drainage will bout $52,570,000.

This system of sewers with the existing 39th Street Conduit was designed arry 5500 cfs of storm runoff, from 27.6 square miles, between 31st et and 87th Street, and east of Racine Avenue to Lake Michigan; or at a off rate of 200 cfs per square mile. These sewers have capacity to carry cfs of sewage, in addition to the 5500 cfs of storm flow, total 5900 cfs.

The sewer outlet capacity of the City of Chicago, past and prospective, ining the South Side sewer system of the Sanitary District, is indicated in tabulation following. Runoff rates are based on the present 221 square es of area within the City of Chicago.

Years	Sewer Outlet Capacity	Runoff cfs per sq. mi.
1900-1910	10,000 cfs	45.5
1920	14,000 cfs	64.0
1930	20,000 cfs	91.0
1940	30,000 cfs	137.0
1950	35,000 cfs	159.0
1957	37,000 cfs	173.0
1960	41,000 cfs	187.0
1970	55,000 cfs	250.0

ewer outlet capacity, thirty years ago, was about ten times the dry ther flow. The outlet sewers now being constructed are designed with caties thirty to fifty times the dry weather flow. This will be reduced by water levels in the Chicago River and the Drainage Canals; but the age capacities in these large relief sewers will be considerable.

These present and prospective sewers are designed to flow their full caties into the rivers or canals at a water level of +1.50 CCD. This water l may reach +4.00 in the Chicago and Calumet Rivers and the effect of would be to reduce the flow capacities of these sewers to 70 or 75 per of their designed capacities; or to reduce the total (1970) capacity from 00 cfs to about 40,000 cfs.

is not expected that the flow capacities of all these outlet sewers could be developed at the same time. The times of concentrations of floods in various sewers and sewer systems range from less than thirty minutes, three hours. However, a 6-inch rain in six hours, such as fell in the e Chicago area on July 12, 1957, could, through this completed sewer em, deliver an amazing amount of storm runoff into the Chicago and met Rivers and the connecting canals. In July 1957 a lot of this went into asement of householders, because of the inadequacy of the local sewers. o doubt these sewers, in particular, the recent auxiliary outlet sewers, been designed on a liberal basis. However, if they have not been greatly

over-designed, a study of how much flood water they can deliver to the outl streams may be a more realistic method of estimating the volume of a futu great flood, than the method outlined on previous pages, of estimating it fro the amounts of water which have been discharged through the Drainage Can in times of past floods.

A study of these outlet sewers, in only the 221 square miles within the c limits of Chicago, indicates the following:

	Percentage of Sewer Capacity	Sewer Capacity - cf		
		Total	75%	5
North of Lawrence Avenue	10%	5,500	4,120	2,
Lawrence Avenue to Lake Street	17%	9,350	7,010	4,
Total North Branch	27%	14,850	11,130	7,
South Branch Chicago River	30%	16,500	12,370	8,
Total Chicago River	57%	31,350	23,500	15,
Drainage Canal, Western Ave. to Harlem Ave.	16%	8,800	6,600	4,
Total to Main Channel	73%	40,150	30,100	20,
Calumet Area, South of 87th Street	27%	14,850	11,130	7,
Total, City of Chicago	100%	55,000	41,230	27,

Storm Flow—Computed from Sewer Capacity

It may be assumed, as quite probable, that at least 50 per cent of the to flow capacity of these sewers will be developed simultaneously, in some fu flood, and that the storm runoff from the 221 square miles within the pres Chicago city limits will be at least 27,500 cfs. To this must be added the storm runoff from about 100 square miles, north of Chicago, in the upper reaches of the North Branch and of the North Shore Channe, including Evanston and the other North Shore towns; and the runoff from about 260 square miles tributary to the Little Calumet River, south of the city. Such runoff may reasonably be calculated at 30 cfs per square mile; and would amount to about 3000 cfs on the North Side, and 7800 cfs in the Calumet region.

The 3000 cfs on the North Side, added to the 20,070 cfs total flow to the Main Channel, from the table above, would indicate a storm flow of at leas 23,070 cfs in the Main Drainage Canal, above Summit. This runoff, from t 308.9 square miles of drainage area above Summit, would be at an average rate of about 75 cfs per square mile.

The 7800 cfs in the Calumet region, added to the 7430 cfs flow to the Calumet-Sage Channel, from the table above, would indicate a storm flow about 15,230 cfs in the Calumet-Sag Channel. This runoff, from the 306 sq miles tributary to the Calumet-Sag Channel, would be at an average rate c about 50 cfs per square mile. Storm flow from the ninety-six square mile in the Sag Valley is ignored in this calculation, since much of this could ha already passed downstream before the peak concentration in the entire Calumet region.

The total storm flow, under this method of calculation, would be 23,070 from the area north of 87th Street plus 15,230 cfs from the Calumet regio

38,300 cfs. This can be compared with the conclusion of 30,000 to 35,000 derived (page 25) from a study of the runoff from past floods.

roduction to Recommendations

The following discussions and tabulation comprise a program of action ıred to increase the outlet capacities for the expected flood waters in the ırs to come. They are by no means the total solution to the problem but y make available sufficient outlet capacities for the rapidly expanding ver programs. It should be understood that these recommendations are sed on the data and experience taken from the operation of the open channel stem and are not meant to solve localized problems due to inadequate sewer pacities. This program is suggested as that part of the total program which l fulfill the outlet capacity needs for flood waters for the next fifty years.

Main Outlet Channels

Since storm runoff in the Chicago and Calumet areas has approached 30,000 on three different occasions in the past ten years (March, 1948, October 54, July 1957), and since building and paving are increasing at very sub-ntial rates, it is apparent that the runoff from a great storm may be 30,000 35,000 cfs, in the not distant future. It is obvious that outlet capacity for ch storm flow should be at least 30,000 cfs.

Chicago is the one city, on the Great Lakes, in the fortunate position of not ving to drink its own sewage, or pass it on for others to drink. However ute it might be, the thought is repugnant. For the past thirty-five years, or ce August 1922, when the Calumet-Sag was opened, Chicago has kept all of sewage out of Lake Michigan, the source of municipal water supply. It has been possible to keep the peaks of flood flows from the Calumet Rivers of the lake, whenever such flood flows have exceeded the hydraulic capaci- of the Calumet-Sag Channel. The same is true of the North Shore Channel, ere storm runoff in recent years has had a great increase, and where the sequent water level has, on many occasions, been high enough to cause w over the restraining walls at Wilmette. On only two occasions (October 54 and July 1957) has it been necessary to permit the Chicago River to flow Lake Michigan, to prevent damage to Loop property, from high water. ese were on occasions of what might be called great storms.

It may be that this policy of Chicago, cherished over the years, of keeping drainage out of its drinking water, may have to be abandoned; at least dur- great storms. It is recommended, however, that sufficient outlet capacity y be provided for storm flows so that relief discharge into Lake Michigan be limited to the peak runoff from great storms.

To that end, a hydraulic study has been made of the present outlet channels of how much their flow capacities should be increased, to meet the needs he near future, of discharge capacity of 30,000 cfs, assuming that any flood v in excess of that amount could be spilled into Lake Michigan. This study, ch should be considered as preliminary only, indicates a need for the owing improvements:

Main Outlet, Drainage Canal, from Sag to Lockport—

Widen present channel, from 160 feet to 300 feet, or more.

This channel would carry about 30,000 cfs, with water surface -3.0 at Sag and -8.0 at Lockport.

Calumet Outlet, Calumet-Sag Channel, from Blue Island to Sag—
Present widening to 225 feet, by Government. This channel should carry about 10,000 cfs, with water surface -3.0 at Sag and +4.0 in Little Calumet River.

Chicago Outlet
Drainage Canal, from Willow Springs to Sag—
Widen present channel, from 160 feet to 230 feet. This channel should carry about 20,000 cfs, with water surface -1.5 at Willow Springs and -3.0 at Sag.

Drainage Canal, from Summit to Willow Springs—
No improvement indicated. Present channel should carry about 20,000 cfs, with water surface +0.1 at Summit and -1.5 at Willow Springs.

Drainage Canal, From Western Avenue to Summit—
Widen present channel 70 feet. The channel should then carry about 20,000 cfs, with water surface +1.5 at Western Avenue and +0.1 at Summit. Perhaps this reach should be improved to the standard dock section, with 240 ft. width between docks; and proper deepenings.

Chicago River and Drainage Canal from Lake Street to Western Avenue—
No improvement indicated. Present channel should carry about 20,000 cfs, with water surface +4.0 at Lake Street and +1.5 at Western Avenue.

These estimates are based on a maximum stage of +4.0 CCD in the Chica and Little Calumet Rivers, of -3.0 at Sag, and a minimum of -8.00 at Lockport. Holding the water level at Sag at about -3.00 is very important. In th March 1948 flood, this level was about +1.60. In the October 1954 flood, it was probably much higher, but the gauge failed. In the July 1957 flood, it w +0.40. These high water levels at Sag cause an unreasonable decrease in th flow capacity of the Drainage Canal above Sag; and in the Chicago River.

Further improvement in the Chicago River and in the Drainage Canal between Damen Avenue and Summit could be easily accomplished by dredging the central 100 feet of the cross-section to a greater depth.

The improvement of these channels, as indicated, would involve the excavation of approximately 7,200,000 cubic yards of rock and about 4,300,000 cubic yards of glacial drift, or earth, at a cost of about $30,000,000. A mo detailed study would result in a better hydraulic balance between the variou channels and sections of channels.

Efforts 1955 to 1957

Since 1955, when the storm water drainage program within The Sanitary District of Chicago, outside the City of Chicago, then estimated to cost $72,000,000, was submitted to the Illinois General Assembly, little has happened to change the picture.

The Metropolitan Sanitary District of Greater Chicago has caused the dredging of 117,325 cubic yards of material from the North Branch of the Chicago River and the North Shore Channel, at a cost of $378,960. This in

enlarged this channel, but such maintenance dredging did restore the nal cross-section and improved conditions for flood flow.

urrently, maintenance dredging of the North Shore Channel, from Foster ue to Wilmette, is under way. Some 250,000 cubic yards of silt and fill o be removed, during the year 1958, at a cost of about $1,110,000, to ore the balance of the North Shore Channel to its original cross-section.

ebris and silt and some shrubbery were removed from the channel of the Plaines River between the old Lockport Controlling Works and Ninth et bridge, in Lockport, and the original channel restored in 1956. Similar is now under way between the Ninth Street bridge and the Elgin, Joliet, Eastern Railway bridge. This will be finished in 1958.

hree additional sluice gates at the old Lockport Controlling Works were pped with operating machinery, 1956-1957, thus adding about 7,500 cfs to lischarge capacity from the Main Drainage Canal, at Lockport. Current- tudies are being made of methods to obtain 10,000 to 12,000 cfs more out- capacity, probably at the Lockport Power House. Outflow capacity was ewhat inadequate during the October 1954 flood, when two of the sluice s were blocked by some loose barges.

he channel of Deep Run is being enlarged between Sixteenth Street bridge, ockport, and its junction with the Main Drainage Canal below the Lockport er House. The improved channel will have a storm flow capacity of 6500 This work will be finished in 1958, at a cost of about $500,000, and ld alleviate the worst flood conditions in Lockport, Illinois.

he City of Chicago continued construction on its long range program of liary Outlet Sewers, presumably at a rate of about $15,000,000 per year. nd issue of $8,000,000, for this work, was approved by the voters, in , leaving about $70,000,000 of the future financing yet to be approved.

Estimates of Cost of Future Work

little has been done toward the alleviation of flood conditions in Cook ty, outside the City of Chicago, that the estimates of costs, submitted in , may well be repeated, with minor modification. One important and cost- em, however, must be added, namely, the enlargement of the Main Outlet nel, at $30,000,000.

he entire future storm drainage program, not yet financed, is outlined in abulation following.

rm Water Drainage Program - February 1958:

Enlargement of Main Drainage Canal	$30,000,000
Improvement of Des Plaines River, from Riverside to north line of Cook County	9,000,000
Improvement of North Branch of Chicago River, north of Lawrence Avenue	4,000,000
Enlargement of North Shore Channel	2,000,000
Enlargement of Stony Creek, and construction of tributary ditches and sewers	8,000,000

Improvement Calumet Union Drainage Ditch	2,000,000
Improvement Feehanville and Weller Ditches	1,000,000
Improvement Thorn Creek	1,000,000
Improvements along Addison and Silver Creeks	4,000,000
Storm water outlets for Posen, Midlothian, Markham, South Blue Island	4,000,000
Storm water outlets, etc. for villages in Grand Calumet and Little Calumet areas	6,000,000
Total Cook County, outside Chicago	$72,000,000
Total City of Chicago, Sewers	70,000,000
Total storm water outlets	$142,000,000

Storm water sewer systems for the various villages in Cook County, esti mated 1955 to cost $31,000,000, are not included in the above estimate. Th improvements of Salt Creek, in DuPage County, estimated 1955 at $5,000,0(is of some importance to the Chicago area, but is omitted.

CONCLUSIONS

From a study of floods in the Chicago area, it is obvious that the presen outlet channels are not adequate to handle the storm runoff during a great flood. No change has been made in the Main Outlet, that is, in the Drainage Canal between Sag and Lockport, since 1900. In that time, the storm runoff from the Chicago and Calumet areas has probably more than tripled. Loca drainage has been and is being improved in many parts of the area, but thes improvements can not be fully effective until the Main Outlet is made adequ

Therefore, the enlargement of the Drainage Canal might be considered a of first importance. This will be particularly true of the stretch of this ca downstream from Sag, since the widening of the Calumet-Sag Channel will, the next four years, cause a tremendous increase in storm flow from the Calumet region. Enlargement of the Drainage Canal is estimated to cost $30,000,000, or about the amount of damage estimated from one great flood in October 1954. Damage from the flood of July 1957 was less, but a figure $10,000,000 has been mentioned. It is probable that the cost of flood protection would be less than the cost of flood damage.

All the projects listed in the program above, except the enlargement of Drainage Canal, are urgently needed for protection against the ordinary flo or that due to a ten-year storm. Residents of some of the communities involved were in pitiful distress during the great floods of 1954 and 1957.

It is not the purpose of this paper to suggest specific improvements for handling storm water flow in any part of the Chicago area; but rather to ou line the problem and furnish background material and factual data which wi assist future students of the subject in arriving at definite recommendation Proper solution of the problem will require a construction program far be the imagination of everybody except a few who have been close enough to it

ze its magnitude. Considerable time, at least ten years, will be required ne physical construction, in an orderly fashion, of the needed works, any method of financing which will not be too burdensome. It is a ram which must necessarily be carried out on an area-wide basis.
nicago is a great city, although it is a young city, only 120 years old. aps it has grown too fast and some of its problems have been neglected. n become the greatest city in the world; and the solution of the storm age problem, now, would enhance that prospect. If the powers that be face that essential issue, and act, future generations will bless them. If procrastinate and postpone such really necessary improvements, future rations will condemn them for lack of courage. It is well known that inde-n is the greatest failing of the average office holder; but there is no time ndecision in a matter so important to a large and populous community as revention of flooding. The longer such matters are deferred, in a grow-ommunity, the greater the cost will be.

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

HYDRAULIC ANALYSIS OF SURGE TANKS BY DIGITAL COMPUTER

Nicholas L. Barbarossa,[1] M. ASCE

SYNOPSIS

It is shown how certain surge problems have been successfully adapted for ution by an IBM 650 computer. Machine results for the Oahe surge system presented and compared with those obtained by independent manual thods of arithmetic integration. Machine-computation time for each of the es discussed averaged between one to one and one-half hours. Sufficient a are given to permit individual checks on machine analysis and computions in a relatively short time.

Computer results are discussed to indicate that treatment of the total blem of transients as one of separable parts in the analysis of surge-tank bility by conventional methods leads to inconclusive results. It appears a comprehensive study by digital computer of the total problem of nsients is feasible, but a machine programming job of great scope will be uired before fruitful results are available.

INTRODUCTION

High-speed, automatic digital computers have the potentiality of opening up horizons of understanding of problem phenomena. Machine solutions will n provide a broader perspective of problem phenomena than can be feasiobtained by manual computations because of the relative rapidity with ch the effects of various factors and assumptions involved in a problem be tested and explored. By demonstrating how this advantage of a digital puter has been actually utilized for solving surge problems in a particular e, additional evidence is provided of the growing usefulness of computers ngineering work.

: Discussion open until September 1, 1959. To extend the closing date one month, written request must be filed with the Executive Secretary, ASCE. Paper 1996 is art of the copyrighted Journal of the Hydraulics Division, Proceedings of the merican Society of Civil Engineers, Vol. 85, No. HY 4, April, 1959.

Chf., Hydrs. Section, U. S. Army Engr. Div., Missouri River, Omaha, Nebr.

The study discussed herein is concerned with mass-oscillation phenomena in a surge system, which includes a pressure conduit and two restricted-orifice type surge tanks in tandem independently connected to the main conduit (Fig. 1). The system supplies water to a reaction-type turbine. Water-hammer effects are not included in the study. Thus, the study is primarily concerned with the solution of surge-tank problems, involving load rejection, load demand, and hydraulic stability of the tanks under small load changes at minimum head.

Given the initial-flow conditions and all other pertinent data, the values of various discharges and heads as a function of time for the unsteady flow conditions within the surge system, resulting from power-load changes, may be determined by conventional methods of arithmetic integration.[1,2]* Application of these methods to surge problems is based upon the fundamental assumption that the compressibility of water and the elasticity of flow boundaries may be ignored. This means that pressure changes in the surge system are transmitted with infinitely high velocities and that water columns in different parts of the system may be regarded as solids in evaluating positive or negative accelerative effects. The same fundamental assumption is used as a basis for development of machine methods. However, since a digital computer is to be employed to obtain solutions, there is no need to make as many other simplifying assumptions as are ordinarily dictated by the time limitations of manual solutions. For example, acceleration heads within the risers, variation of friction factors with changing Reynolds Number, tee losses, and finer time increments in the step-by-step process may all be introduced with only comparatively small increase in the machine time required for problem solutions.

A list of the physical relationships required for machine solution of surge problems associated with the system depicted on Fig. 1 and the sign convention that has been used are given in Appendix III.

Basis of Analysis and Computation

Head and Discharge Relationships

Based upon the fundamental assumption of instantaneous transmission of pressure changes, application of the energy principle to the upstream conduit (Fig. 1) for any instant of time yields**

$$h_{a_1} = E_R - P_1 - h_{L_1} \qquad (1)$$

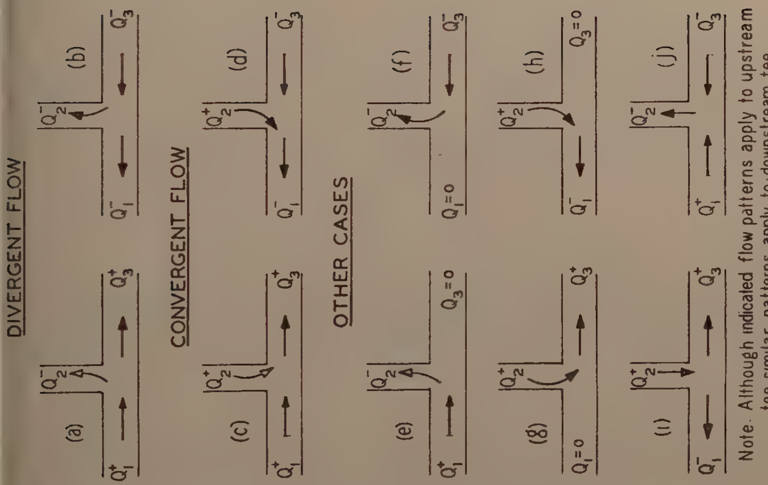
The velocity head for the upstream conduit is included in h_{L_1} for convenience; also included are all form and surface-resistance losses between the reservoir and point (1). Relationships similar to Eq. (1) for other parts of the surge system are given in Appendix III.

All acceleration heads are defined by the conventional, one-dimensional formula

$$h_a = \frac{L}{gA} \frac{dQ}{dt} \qquad (2)$$

*The numbers in parentheses indicate references listed in Appendix II.

**The symbols used are defined in Appendix I.



Note. Although indicated flow patterns apply to upstream tee, similar patterns apply to downstream tee.

FIGURE 2 – FLOW AT THE TEES

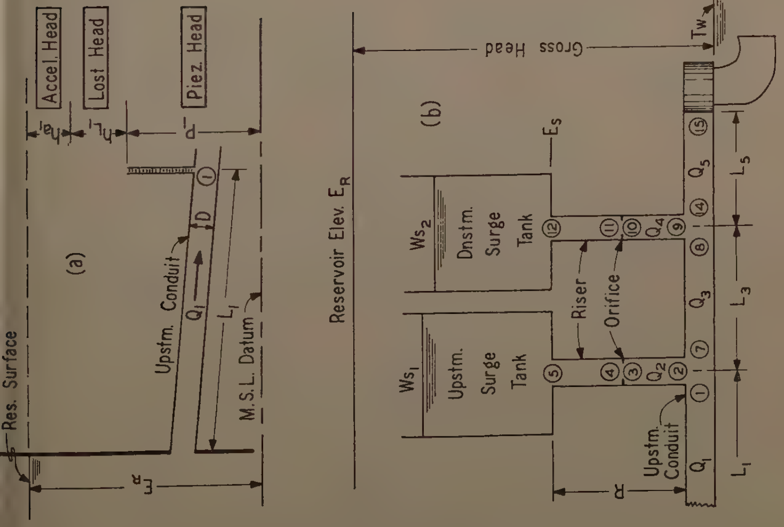


FIGURE 1 – DEFINITION SKETCH

The sign of h_a will depend upon the sign of the derivative $\frac{dQ}{dt}$.

By appropriate use of Eq. (2), Eq. (1) may be written in the form

$$\frac{dQ_1}{dt} = \frac{gA}{L_1}(E_R - P_1 - h_{L_1}) \tag{3}$$

Similar relationships may be developed for other Q's. The diameter of the main conduit has the same value for the lengths L_1, L_3, and L_5. All corresponding dimensions of the upstream and downstream risers and surge tanks are the same.

The net head on the turbine is obtained from the relationship,

$$H = P_{15} + \frac{V_5^2}{2g} - Tw \tag{4}$$

In Eq. (4), the comparatively small velocity-head lost in the tail-race is ignored. Moreover, Tw is assumed to be constant during any one solution.

Application of the continuity principle at each tee requires that

$$Q_1 + Q_2 = Q_3 \tag{5}$$

$$Q_3 + Q_4 = Q_5 \tag{6}$$

at each instant, even when flow reversals take place. Differentiation of Eq. (5) gives

$$\frac{dQ_1}{dt} + \frac{dQ_2}{dt} = \frac{dQ_3}{dt} \tag{7}$$

A similar expression may be obtained by differentiating Eq. (6). Equations of the form indicated by (7) are useful for eliminating the derivatives from equations of the form indicated by (3). Actual use is shown in Appendix III.

The relationships between the water surfaces in the surge tanks and the instantaneous flows in or out of the tanks are obtained from the consideration that inflow minus outflow equals the change in storage during dt:

$$\frac{d(Ws_1)}{dt} = -\frac{Q_2}{A_f} \tag{8}$$

$$\frac{d(Ws_2)}{dt} = -\frac{Q_4}{A_f} \tag{9}$$

For cases when the water surface in either tank drops below the level E_s, A_f in either Eq. (8) or (9) is replaced by A_r.

Head Losses

All surface-resistance losses h_f in feet are evaluated by the Darcy-Weisbach equation,

$$h_f = f\,\frac{L}{D}\,\frac{|Q|Q}{2gA^2} \tag{10}$$

This equation is applied not only to the main conduit but also to the risers. Surface-friction losses within the surge tanks are ignored because, ordinarily they are negligibly small.

In some installations a part of the total length L_1 of the upstream conduit ll be lined with concrete and the remainder with steel. Provision for this s been made in the machine program. Let L_c = length that is lined with ncrete and L_s = length that is lined with steel, so that $L_1 = L_c + L_s$. Desig-ting f_c and f_s as the friction factors for the concrete and steel lengths, re-ectively, and utilizing Eq. (10), the total surface-resistance losses from the servoir to point (1) are given by

$$h_{f_1} = (f_c L_c + f_s L_s) \frac{|Q_1| Q_1}{2gDA^2} \tag{11}$$

Except for the tee losses, all form (or shape) losses h_s in feet are evaluat- by an equation of the type,

$$h_s = K \frac{|Q| Q}{2gA^2} \tag{12}$$

which K is assumed to be constant for a particular flow-boundary configu-tion. Eq. (12) indicates that h_s will have the same sign as Q. The total ad loss* due to various boundary forms from the reservoir to point (1) is ven by

$$h_{s_1} = (K_v + K_{tr} + K_e + K_s + K_{ts} + K_{hb} + K_{vb} + K_{ot}) \frac{|Q_1| Q_1}{2gA^2} \tag{13}$$

rictly, all K's in Eq. (13) will have one set of values when Q_1 is positive and ll have a different set of values when Q_1 is negative (flow toward the reser-ir). In this study the K's are assigned one set of values and are assumed remain constant during any one machine run. (If deemed necessary, the chine program can be modified to allow the K's to vary with the direction Q_1 and also with the magnitude of the velocity head.)

Combining Eqs. (11) and (13), the total head loss from reservoir to point is

$$h_{L_1} = \left[K_1 + K_1' (f_c L_c + f_s L_s)\right] |Q_1| Q_1 \tag{14}$$

Head losses (other than tee losses) for other parts of the main conduit are nilarly determined, using analogous notation (Appendix III).

In evaluation of the losses in the risers and at points (5) and (12), attention st be given to the direction of Q_2 and Q_4 and to whether or not the water rfaces Ws_1 and Ws_2 are above or below the elevation E_s at any instant under nsideration. The total head loss for the upstream surge tank and riser stem is

$$h_{L_2} = \left(C_r + K_2 f_2 + \frac{1}{2gC_o^2 A_o^2}\right) |Q_2| Q_2 \tag{15}$$

The first term in Eq. (15) evaluates: (a) the loss of velocity head when the er is delivering water to the surge tank; (b) the velocity head in the riser the entrance loss when the surge tank is delivering water to the riser. e second term in Eq. (15) evaluates the surface-friction loss in the riser.

rictly, h_{s_1} is more than a head loss, since it includes the velocity head in e upstream conduit. K_v is equal to one in all cases.

Eqs. (III-27) and (III-28) in Appendix III indicate how C_r and K_2 are evaluate In the two expressions for K_2 (Eq. (III-28)) allowance is made as to whether or not the total friction boundary has a length R or something less than R. The third term in Eq. (15) determines the head drop across the orifice and i based upon the standard-orifice formula. In this study C_o is taken as being constant during any one machine run. Actually, even if the orifice may be considered a thin-plate orifice as such, it is likely that C_o will vary as Q_2 changes owing to the condition that the orifice plate is probably not sufficien distant from major flow disturbances in most installations. However, C_o m be easily introduced into the machine program as a function of appropriate variables, if desired and if data are available for realistic evaluation. Othe wise, C_o may be varied from one run to another to see what effect different values have on the hydraulic transients, as has been done in the Oahe study. Head losses h_{L_4} in the downstream riser and surge tank are obtained in the same manner as used to obtain h_{L_2}.

Tee Losses

In determining the tee losses at any instant of time, the three Q's at eacl tee must be known in both magnitude and direction at that instant. Evaluati of the tee loss will then depend upon which of the possible flow patterns indi cated on Fig. 2 applies at that instant. Provisions have been made in the machine program for evaluating the tee losses at each instant of time for al flow patterns shown on Fig. 2 except the last two (i and j). (When flow patterns 2(i) and 2(j) occur, the tee loss is assumed to be zero.) For these two cases, it is uncertain that appropriate data for evaluation exist. Should data be available, the program could be readily modified to accommodate these two cases. Data are available[4] to evaluate cases (a) through (h) on Fig. 2. Obviously, cases (e) through (h) are special cases of (a) through (d)

The tee losses T in feet are determined by equations of the general forn

$$T = C_t \frac{V^2}{2g} \qquad (\text{}$$

wherein, C_t is a variable coefficient, which depends upon the discharge rati $\frac{Q_2}{Q_1}$ or $\frac{Q_2}{Q_3}$ at the upstream tee ($\frac{Q_4}{Q_3}$ or $\frac{Q_4}{Q_5}$ at the downstream tee) and upon whether the flow is divergent or convergent. The velocity head to be insert in Eq. (24) is either $\frac{V_1^2}{2g}$ or $\frac{V_3^2}{2g}$ at the upstream tee ($\frac{V_3^2}{2g}$ or $\frac{V_5^2}{2g}$ at the dow stream tee) depending upon whether the main conduit flow is directed towar (+) or away (-) from the turbine. In Eq. (16) C_t is replaced by C_D and C_D' f divergent flow and by C_c and C_c' for convergent flow. Four sets of coeffici are required in order to obtain the piezometric-head changes at each tee fo all cases considered. (Table 5 has been utilized for numerical evaluation when $D_r/D = 2/3$.) The coefficients include allowances for changes in veloc head from conduit to riser and vice versa. The details of tee-loss determi nation are given in a summarization in Appendix III.

With reference to Fig. 1, if the tee losses are ignored, the piezometric heads at points (1), (2), and (7) are assumed to be equal; that is, $P_1 = P_2 =$ These three heads differ only when the tee losses are introduced. Thus, fo example,

$$P_1 - P_2 = T_{1,2} = C_D \frac{V_1^2}{2g} \tag{17}$$

$$P_1 - P_7 = T_{1,7} = C_D' \frac{V_1^2}{2g} \tag{18}$$

both of these equations (applicable to case (a) on Fig. 2) the sequence of riting the subscripts for T is important from the standpoint of avoiding con- ısion in signs. Thus, $T_{1,2}$ indicates $P_1 - P_2$ and not $P_2 - P_1$. The same easoning applies to the downstream tee.

In all instances of unsteady flow when the tee losses are taken equal to or re zero, the total energy heads at the tees are not balanced because of the ıstantaneous difference in velocity heads between main conduit and riser etween points (1) and (2) on Fig. 2, for example). (The number of times that ıis would occur during one total solution when the tee losses are evaluated at ll other instances during that same solution is relatively small.) This un- alanced condition may be assumed to be taken care of by orifice effects in ıe riser, that is, by the orifice coefficient. If deemed necessary, special pro- isions can be incorporated in the machine program to take care of the elocity-head differences when T = 0. In this study, such provisions have not een made, because over-all results for the Oahe project, at least, would not e significantly affected.

ate-Time and Turbine-Performance Curves

Transient effects within the hydraulic system are established by turbine icket-gate movements. In order for the program to work successfully, the ate opening G in per cent must be known at each instant of time during the ıitial interval of governor action, after which the gate opening is assumed to emain constant. This statement is only partially true for surge-tank stabili- studies. (Discussion of what is involved in stability studies is presented in nother section of the paper.) The gate-time curve is given by

$$G = F_g(t), \tag{19}$$

which, F_g = a function. It is not necessary that this be a linear function of me. Any single-valued function of time may be used, and it may include ead and cushion time (for opening and closing of gates, respectively.) Such ırves are usually furnished by the manufacturer of the turbine.

In addition, turbine-performance curves are required. They enter the rogram as a table involving three variables, whose inter-relationship in eneral mathematical form is expressed as

$$H = F(G, Q_5), \tag{20}$$

which F is a function. For carrying out solutions for full-load rejection at aximum turbine design head H_{max} and for full-load demand at minimum rbine design head H_{min}, it will usually be necessary to have tabular values G and Q_5 for values of H well above H_{max} and of H well below H_{min}. Other- ise, the computer will stop because of artificial bounds imposed by lack of bular values in regions of computation which the analyst did not foresee or erlooked. (What this means is that when the computer reaches the end of a ble and it has no instructions on what to do next, it stops.)

To perform surge-tank stability studies along conventional lines, the following set of turbine curves is utilized instead of the family of curves indicated by Eq. (20);

$$H = F_q(Q_5)_{Bhp} = \text{Constant} \tag{21}$$

in which F_q is a function and Bhp = brake horsepower. This equation simply gives the relationship between H and turbine discharge Q_5 when the turbine power output is constant and can include the effects of varying efficiency. Th use that has been made of such curves and the difficulties experienced in the computer study are discussed below.

Machine Computation

With the aim of keeping machine-programming details to a practical minimum herein, Fig. 3 has been especially prepared for the purposes of this paper. The chart simply depicts the main elements of problem solution but not the actual sequence of machine operations.* To show the actual sequence of machine operations would require considerable detail, the inclusion of which would tend to obscure the main elements of the problem. Fig. 3 applie to problem solutions that start from steady-state conditions, but the detailed machine program is not so limited. The equation numbers shown in some of the boxes refer to the relationships given in Appendix III. It is suggested tha the reader first examine that appendix before proceeding with the remainder of the paper.

In box 6 (Fig. 3) all lost heads are computed from the steady-state $Q = Q_1$ $Q_3 = Q_5$ and data given in boxes 2, 3, and 4, and tailwater level is adjusted to the value it must have for all heads to balance exactly within the machine for the given reservoir elevation E_R (box 2). Alternatively, the tailwater level may be kept constant, and E_R may then be adjusted for balancing of steady-state heads. This balancing process insures that the computer starts off wit a totally consistent set of values for the variables involved.

Once steady-state conditions are balanced within the machine, a transient is set up by applying the initial time interval Δt (box 5), which fixes the new gate setting from $G = F_g(t)$ in box 4. The new G in turn fixes the new H from the turbine-performance curves (box 10), using the value of Q_5 at the beginning of the time interval. As a result, all other variables are changed, and the machine will compute their values at each step as the transient develops. The initial Δt is arbitrarily selected by the analyst and can be as small as seems warranted. Values as low as 0.01 second have been used to start the transient; in most cases, initial $\Delta t = 0.10$ has been used. The computer can be instructed through control information (box 5) to use Δt values of 0.1 up to a total time of $t = 1.00$ and to use time intervals of one second thereafter up to, say, 80 seconds, after which it would use 5-second intervals until the tota time for the machine run (given in box 5) is reached. These figures are mer ly illustrative. Any reasonable set of time intervals may be used, including possible use of a constant Δt to the end of the run. However, the method of solving differential equations (discussed below) will not work expeditiously if the assigned time intervals for step solutions are too large.

*Complete details, including all flow charts and the machine codes, are give in the machine program itself, which is being prepared as a separate document for distribution on a limited basis.

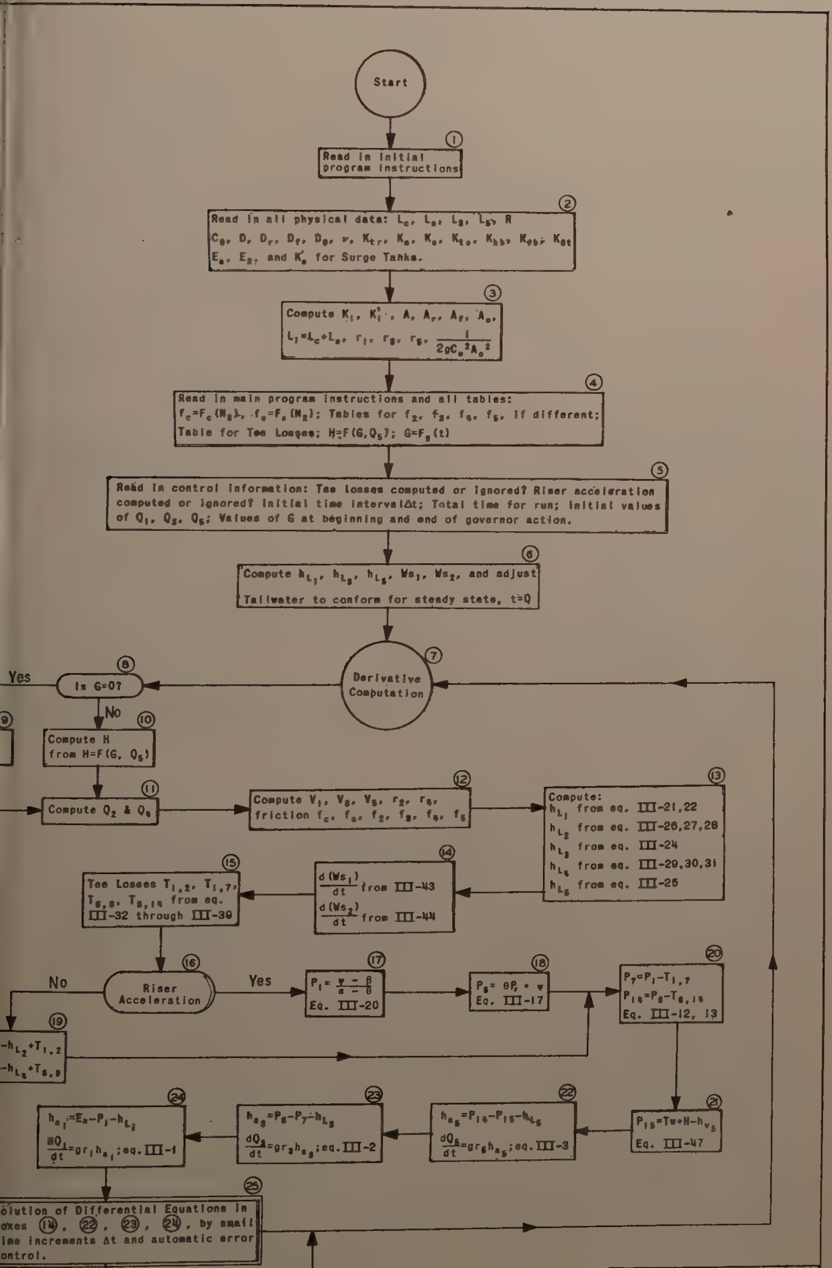


Figure 3

ABBREVIATED FLOW DIAGRAM FOR SOLUTIONS TWO RESTRICTED-ORIFICE SURGE TANKS IN TANDEM

With reference to Fig. 3, computations are carried out in the loop, boxe through 25 and back to 7 (but not precisely in the sequence shown), with pr out of results in box 26 occurring whenever all the conditions for solution each time step are satisfied. The computer may work around this loop ma times during any time step before print-out of results occurs. The compu may print out all or any number of the quantities indicated in box 26. In b 9, the indicated relationship $r_5 = 0$ is, of course, never true. This is simp a trick which is used to save some programming steps for rejection cases when $G = 0$, $Q_5 = 0$, and $\frac{dQ5}{dt}$ must equal zero in box 22. It will be noted tha the computer determines the head losses (box 13) and tee losses (box 15) a every step. By manual methods, this would entail a tremendous amount of work. To calculate all the h_L's at each step requires the determination of five Reynolds Numbers and linear interpolations in the tables for six frict factors (box 4). Tee losses are evaluated in accordance with all the con-ditions specified in Appendix III and by linear interpolation in a table havi four sets of coefficients. This degree of refinement in surge-problem co tations is practicable because the machine costs involved are comparative small.

The choice of whether or not to include riser acceleration (box 16) is n necessary in future applications. The option was incorporated in this program in order to give a direct indication of what effects this item woul have on final results. An option on computation of tee losses has also bee incorporated in the machine program, but the option is not indicated on Fi (box 15). Here again, in future applications, there would be no necessity f ignoring tee losses. As long as the form of the tee-loss equations and the method of evaluation do not change, any set of numerical values may be ut ized for the tee-loss coefficients in the machine program.

Machine Solution of Differential Equations

Knowing the values of the primary variables Q_1, Q_3, Q_5, Ws_1, and Ws_2 the beginning of a time step, $\Delta t = t_2 - t_1$, the derivatives of these five var ables (boxes 14, 22, 23 and 24 in Fig. 3) may be readily determined for ti t_1. (Q_2 and Q_4 are not designated as primary variables, since their valu are easily obtained by algebraic subtractions through use of continuity Eq (5) and (6)). The values of the primary variables at the end of the time ste t_2, are obtained by a process of high-order approximations.(10)

For ease in writing let y equal any one of the five primary variables fo the purposes of this section of the paper. Then the values of any one of th quantities at the beginning, mid-point, and end of the time step Δt are des nated as y_1, $y_{1.5}$, and y_2, respectively. Roman numerals are utilized to d nate approximate values of $y_{1.5}$ and y_2. Thus, y_2^{I} indicates the first appr mation of y_2, y_2^{II} the second approximation, etc. At time t_1, y_1 and $\frac{dy_1}{dt}$ ar known. A first approximation is made by utilizing the value of the deriva at t_1 to yield

$$y_2^{I} = y_1 + \frac{dy_1}{dt} \Delta t .$$

Having a first approximation of the five primary variables, as indicated b Eq. (22), new values (temporary) of their five derivatives are then compu

tly the same way as was done at time t_1; these new derivatives are desig-
d $\frac{dy_2^I}{dt}$. The next step yields a better approximation of y_2 by making use
ne average of the two derivatives now at hand, so that

$$y_2^{II} = y_1 + \frac{\Delta t}{2} \left(\frac{dy_1}{dt} + \frac{dy_2^I}{dt} \right). \tag{23}$$

ing set y_2^{II} as given by Eq. (23) aside, the computer cuts the original time
rval in half and determines $y_{1.5}^{II}$ in exactly the same manner as y_2^{II} was
ulated, with the result that

$$y_{1.5}^{I} = y_1 + \frac{dy_1}{dt} \frac{\Delta t}{2}, \text{ and} \tag{24}$$

$$y_{1.5}^{II} = y_1 + \frac{\Delta t}{4} \left(\frac{dy_1}{dt} + \frac{dy_{1.5}^I}{dt} \right) \tag{25}$$

Still another approximation for y_2 is obtained by applying the same method
omputation to the second half of the time interval (from $t_{1.5}$ to t_2), so

$$y_2^{III} = y_{1.5}^{II} + \frac{dy_{1.5}^{II}}{dt} \frac{\Delta t}{2}, \text{ and} \tag{26}$$

$$y_2^{IV} = y_{1.5}^{II} + \frac{\Delta t}{4} \left(\frac{dy_{1.5}^{II}}{dt} + \frac{dy_2^{III}}{dt} \right). \tag{27}$$

final values of all primary variables at time t_2 are taken as*

$$y_2 = y_2^{IV} + \frac{y_2^{IV} - y_2^{II}}{3} \tag{28}$$

before these final values are assigned to y_2, relative error tests are
le. The relative error is defined as

$$R_y = \frac{y_2^{IV} - y_2^{II}}{3y_2^{IV}}. \tag{29}$$

$R_y| > \epsilon_1$, a pre-assigned margin of relative error, the computer cuts the
inal time interval Δt in half and goes through the whole procedure (em-
ied in Eqs. (22) through (29)) all over again. The machine will repeat this
cess until $|R_y| < \epsilon_1$. (For the Oahe project, ϵ_1 was given a value of 10^{-3}.)
prevent the computer from taking unnecessarily small time increments,
machine is instructed to double the original time interval whenever
$| < \epsilon_2$, another pre-assigned margin of relative error. (For the Oahe
ect ϵ_2 was assigned a value of $\epsilon_1/100$, that is 10^{-5}.) By the procedure
outlined, solutions are advanced time step by time step and automatic
or control is achieved at each step.

e factor three in Eq. (28) is based upon theoretical considerations.[10]

The necessity for such high-order approximations for solutions of differential equations has not been investigated, because the machine time required per total solution is reasonable and because office time could not be spared for the mathematical explorations that would be required. It is believed that the machine method of solving differential equations for surge problems, as discussed herein, can also be applied profitably to machine solutions of a variety of other engineering problems.

Application of Machine Methods to Oahe Surge System

Conditions and Data for Numerical Solutions

Oahe Dam, located about six miles upstream of Pierre, South Dakota, is key project of the system of multiple-purpose reservoirs on the Missouri River.(5,6) Each of the scheduled seven generator units at Oahe will have a nameplate capacity of 85,000 kilowatts, and hydro-power will be delivered to the generators by Francis-type turbines. The hydraulic surge system pertaining to any one of the seven units is schematically shown on Fig. 1. The length of the upstream conduit L_1 is different for each unit due to physical layout. Machine results have been obtained only for unit number 6, for which L_1 = 3,820 feet in all cases, except cases Ie and IIe in Tables 6 and 7. These two tables and Table 8 give the initial conditions (steady state) and other pertinent data upon which manual and all machine computations for full-load rejection, full-load demand, and surge-tank stability analysis are based. References to manual computations in all tables and figures herewith apply to those made independently by engineers of Sverdrup and Parcel, Inc., a firm under contract with the Omaha District, Corps of Engineers, for design of the Oahe power plant and surge tanks.

All computations are based upon: surge tank diameters of 70 feet with no interconnection between tanks; riser diameters of 16 feet; main conduit and penstock diameter of 24 feet; riser height R of 81 feet; orifice diameters of 12.33 feet; a distance between risers L_3 of 40 feet (except for cases Ie and IIe); and other data given in Tables 6, 7, and 8. With L_3 = 40 feet, the surge tanks will, of course, not fit concentrically over the risers. Hence, for cases Ie and IIe the distance L_3 was doubled in order to permit a concentric arrangement of surge tanks and risers, if desired. From a hydraulic stability standpoint, the distance between tanks should probably be as small as practicable. The eccentric arrangement with L_3 = 40 feet has been adopted for the Oahe project.

All friction factors for both manual and machine computations have been evaluated from data given in Tables 1, 2, and 4. When $N_R < 10^5$, friction losses for the smallest diameter (16 feet) are negligibly small, and hence, values of f are given in the tables for $N_R < 10^5$. Table 5 has been used to evaluate all tee losses for computer solutions. Manual computations in all cases are based upon ignoring acceleration effects in the risers and in the length L_5, but tee losses have been evaluated using Vogel's data.(4) It will be noted from Tables 6, 7, and 8 that L_1 for manual computations was 3,580 feet instead of 3,820 feet and that a few other items for these computations differ slightly from those used for machine solutions. The reason for this is that computer runs were made at a much later data, after some Oahe design changes had been effected. Other conditions and data as well as results are discussed below.

TABLES 1, 2 & 4

Friction Factors f

BLE 1*	TABLE 2**		TABLE 4†	
f	Reynolds Number N_R	f	Reynolds Number N_R	f
0.0182	10^5	0.0186	10^5	0.0196
0.0144	2×10^5	0.0167	2×10^5	0.0186
0.0118	3×10^5	0.0158	3×10^5	0.0176
0.0098	5×10^5	0.0147	5×10^5	0.0165
0.0091	7×10^5	0.0143	7×10^5	0.0161
0.0082	10^6	0.0138	10^6	0.0158
0.0074	2×10^6	0.0132	2×10^6	0.0155
0.0070	4×10^6	0.0128	4×10^6	0.0152
0.0067	6×10^6	0.0127	6×10^6	0.0152
0.0064	10^7	0.0125	10^7	0.0152
0.0059	10^8	0.0125	10^8	0.0152

Von Karman-Prandtl Curve for hydrodynamically smooth
dar ies.

d upon Relative Roughness $e/D = 1.25 \times 10^{-4}$

d upon Relative Roughness $e/D = 3.3 \times 10^{-4}$

es in Tables 2 and 4 taken from U. S. Bureau of Reclamation
graph #7, Figure 4.

ABBREVIATED
TABLE 3

OAHE POWER PLANT
TURBINE PERFORMANCE

Values of Discharge in C.F.S. Versus Gate Opening and Net Head

G-Gate Opening in %	Net Head on Turbine H in Feet							
	70	110	120	130	160	185	205	240
0	0	0	0	0	0	0	0	0
5	276	370	393	412	470	516	549	588
10	551	740	786	824	940	1031	1097	1176
15	827	1110	1178	1237	1410	1547	1646	1764
20	1102	1480	1571	1649	1880	2062	2194	2351
25	1378	1850	1964	2061	2350	2578	2743	2939
30	1653	2220	2357	2473	2820	3093	3291	3527
35	1929	2590	2749	2886	3290	3609	3840	4115
40	2204	2960	3142	3298	3760	4124	4396	4725
45	2480	3330	3535	3710	4230	4640	4936	5275
50	2800	3705	3910	4110	4665	5092	5398	5800
55	3155	4065	4280	4490	5090	5632	5840	6260
60	3460	4430	4655	4870	5485	5948	6278	6740
65	3745	4775	5010	5230	5845	6322	6682	7235
70	4030	5135	5375	5600	6215	6692	7050	7585
75	4355	5430	5670	5900	6545	7048	7428	8015
80	4600	5750	5985	6215	6875	7390	7778	8330
85	4900	6035	6285	6525	7190	7705	8095	8715
90	5195	6280	6530	6770	7475	8025	8425	9015
95	5450	6490	6735	6975	7685	8250	8677	9355
100	5705	6670	6910	7160	7875	8458	8905	9665

Adapted from Omaha District Curves, Model 734-H, dtd 20 November 1957.

TABLE 5

Tee Loss Coefficients*

$D_r/D = 2/3$

Discharge Ratio $\frac{Q_{riser}}{Q_{conduit}}$ **	Diverging Flow			Converging Flow		
	C_D	C_b	$C_D - C_b$	C_c	C'_c	$C'_c - C_c$
0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.1	0.15	-0.24	0.39	0.43	0.50	0.07
0.2	0.45	-0.47	0.92	0.68	0.83	0.15
0.3	0.90	-0.60	1.50	0.86	1.12	0.26
0.4	1.47	-0.69	2.16	0.97	1.38	0.41
0.5	2.19	-0.75	2.94	1.07	1.62	0.55
0.6	3.00	-0.75	3.75	1.10	1.82	0.72
0.7	4.00	-0.77	4.77	1.10	2.00	0.90
0.8	5.20	-0.75	5.95	1.05	2.17	1.02
1.0	7.85	-0.65	8.50	0.70	2.45	1.75
Use For Tee Losses	$T_{1,2}$ $T_{8,9}$	$T_{1,7}$ $T_{8,14}$	$T_{1,2}$ $T_{8,9}$	$T_{1,2}$ $T_{8,9}$	$T_{1,7}$ $T_{8,14}$	$T_{1,2}$ $T_{8,9}$

* Prepared by Omaha District, CE, from Vogel's data (4).

** Discharge ratios may be either $\frac{Q_2}{Q_1}$, $\frac{Q_2}{Q_3}$ or $\frac{Q_4}{Q_3}$, $\frac{Q_4}{Q_5}$.

OAHE POWER PLANT SURGE TANK ANALYSIS
Two Restricted-Orifice Surge Tanks in Tandem
TURBINE UNIT NO. 6

Table 6 FULL LOAD REJECTION

	MANUAL COMP.	CASE 1	CASE 1b	CASE 1c	CASE 1d	CASE 1e
Initial Conditions						
Gate Opening %	70.0	66.3	66.3	66.3	66.3	66.3
Net Head on Turbine, ft.	203.0	203.0	203.0	203.0	203.0	203.0
Turbine Q_5, cfs	6800	6745	6745	6745	6745	6745
Brake Horsepower						
New Conditions						
Desired Brake Horsepower						
Gate Opening %	0	0	0	0	0	0
Reservoir Ws Elev., E_R	1645.0	1645.0	1645.0	1645.0	1645.0	1645.0
Tailwater Elev.	1436.9	1436.6	1436.6	1436.6	1436.6	1436.6
Elev. Surge Tank Base, E_s	1515.0	1515.0	1515.0	1515.0	1515.0	1515.0
Elev. of Distributor, E_t	1422.0	1422.0	1422.0	1422.0	1422.0	1422.0
Lengths in Feet						
Upstrm Conduit, Total L_1	3580.0	3820.0	3820.0	3820.0	3820.0	3798.5
Upstrm Conduit, conc., L_c	1300.0	1292.0	1292.0	1292.0	1292.0	1292.0
Upstrm Conduit, Steel, L_s	2280.0	2528.0	2528.0	2528.0	2528.0	2506.5
Between Tanks, L_3	40.0	40.0	40.0	40.0	40.0	80.0
Dnstrm Tank to Turbine, L_5	66.0	66.0	66.0	66.0	66.0	42.5
Riser, R	81.0	81.0	81.0	81.0	81.0	81.0
Diameters in Feet						
Surge Tank, D_f	70.0	70.0	70.0	70.0	70.0	70.0
Riser, D_r	16.0	16.0	16.0	16.0	16.0	16.0
Orifices, D_o	12.33	12.33	12.33	12.33	12.33	12.33
Penstock, D	24.00	24.0	24.0	24.0	24.0	24.0
Governor Time, Seconds	3.50	3.31	3.31	3.31	3.31	3.31
Basis of Computation						
Accel. Head in Riser	NC	C	NC	C	C	C
Accel. Head in L_5	NC	C	C	C	C	C
Head Loss @ Tees	C	C	C	C	C	C
Head Loss @ Orifice	C	C	C	C	C	C
Friction in Riser, R	C	C	C	C	C	C
Loss, Tank to Riser	C	C	C	C	C	C
Friction Factor f, varying	C	C	C	C	C	C
Other Data						
Temperature, °F	38.0	38.0	38.0	38.0	38.0	38.0
$\nu \times 10^5$ (kin. viscosity)	1.71	1.73	1.73	1.73	1.73	1.73
Surge Tank, K_t		0.50	0.50	0.50	0.50	0.50
Form Losses in L_1	Min.	Min.	Min.	Min.	Min.	Min.
Trashrack, K_{tr}	0.03	0.03	0.03	0.03	0.03	0.03
Entrance, K_e	0.03	0.03	0.03	0.03	0.03	0.03
Gate Slots, K_s	0.01	0.01	0.01	0.01	0.01	0.01
Transitions, K_{ts}	0.03	0.03	0.03	0.03	0.03	0.03
Horiz. Bends, K_{hb}	0.02	0.02	0.02	0.02	0.02	0.02
Vert. Bends, K_{vb}	0.03	0.03	0.03	0.03	0.03	0.03
Other, K_{ot}	0.20	0.20	0.20	0.20	0.20	0.20
Vel. Head, K_v	1.00	1.00	1.00	1.00	1.00	1.00
Total; ΣK	1.35	1.35	1.35	1.35	1.35	1.35
Orifice Ratio D_o/D_r	0.771	0.771	0.771	0.771	0.771	0.771
Orifice Coeff., C_o	0.70	0.70	0.70	0.80	0.60	0.70
Friction Losses						
Smooth-pipe, use	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1
Concrete, use	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1
Concrete e/D x 10^4						
Steel, use	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1
Steel, e/D x 10^4						
Tee Losses						
Ratio D_r/D=2/3, use		Tab. 5	Tab. 5	Tab. 5	Tab. 5	Tab. 5

Table 7 FULL LOAD DEMAND

	MANUAL COMP.	CASE 11	CASE 11a	CASE 11b	CASE 11d	CASE 11g
Initial Conditions						
Gate Opening %	0	5.0	5.0	5.0	5.0	5.0
Net Head on Turbine, ft.	120.0	118.79	118.79	118.79	118.79	118.79
Turbine Q_5, cfs	0	390.0	390.0	390.0	390.0	390.0
Brake Horsepower						
New Conditions						
Desired Brake Horsepower						
Gate Opening %	90.0	100.0	100.0	100.0	100.0	100.0
Reservoir Ws Elev., E_R	1540.0	1540.0	1540.0	1540.0	1540.0	1540.0
Tailwater Elev.	1420.0	1421.17	1421.17	1421.17	1421.17	1421.17
Elev. Surge Tank Base, E_s	1515.0	1515.0	1515.0			
Elev. of Distributor, E_t	1422.0	1422.0	1422.0	1422.0	1422.0	1422.0
Lengths in Feet						
Upstrm Conduit, Total L_1	3580.0	3820.0	3820.0	3820.0	3820.0	3798.5
Upstrm Conduit, conc., L_c	1300.0	1292.0	1292.0	1292.0	1292.0	1292.0
Upstrm Conduit, Steel, L_s	2280.0	2528.0	2528.0	2528.0	2528.0	2506.5
Between Tanks, L_3	40.0	40.0	40.0	40.0	40.0	80.0
Dnstrm Tank to Turbine, L_5	66.0	66.0	66.0	66.0	66.0	42.5
Riser, R	81.0	81.0	81.0	81.0	81.0	81.0
Diameters in Feet						
Surge Tank, D_f	70.0	70.0	70.0	70.0	70.0	70.0
Riser, D_r	16.0	16.0	16.0	16.0	16.0	16.0
Orifices, D_o	12.33	12.33	12.33	12.33	12.33	12.33
Penstock, D	24.0	24.0	24.0	24.0	24.0	24.0
Governor Time, Seconds	5.0	5.0	10.0	5.0	5.0	5.0
Basis of Computation						
Accel. Head in Riser	NC	C	C	NC	C	C
Accel. Head in L_5	NC	C	C	C	C	C
Head Loss @ Tees	C	C	C	C	C	C
Head Loss @ Orifice	C	C	C	C	C	C
Friction in Riser, R	C	C	C	C	C	C
Loss, Tank to Riser	C	C	C	C	C	C
Friction Factor f, varying	C	C	C	C	C	C
Other Data						
Temperature, °F	38.0	38.0	38.0	38.0	38.0	38.0
$\nu \times 10^5$ (kin. viscosity)	1.71	1.73	1.73	1.73	1.73	1.73
Surge Tank, K_t	0.50	0.50	0.50	0.50	0.50	1.00
Form Losses in L_1	Max.	Max.	Max.	Max.	Max.	Max.
Trashrack, K_{tr}	0.10	0.10	0.10	0.10	0.10	0.10
Entrance, K_e	0.08	0.08	0.08	0.08	0.08	0.08
Gate Slots, K_s	0.08	0.08	0.08	0.08	0.08	0.08
Transitions, K_{ts}	0.08	0.08	0.08	0.08	0.08	0.08
Horiz. Bends, K_{hb}	0.04	0.04	0.04	0.04	0.04	0.04
Vert. Bends, K_{vb}	0.10	0.10	0.10	0.10	0.10	0.10
Other, K_{ot}	0.30	0.30	0.30	0.30	0.30	0.30
Vel. Head, K_v	1.00	1.00	1.00	1.00	1.00	1.00
Total; ΣK	1.78	1.78	1.78	1.78	1.78	1.78
Orifice Ratio D_o/D_r	0.771	0.771	0.771	0.771	0.771	0.771
Orifice Coeff., C_o	0.70	0.70	0.70	0.70	0.60	0.70
Friction Losses						
Smooth-pipe, use						
Concrete, use	Tab. 2	Tab. 2	Tab. 2	Tab. 2	Tab. 2	Tab. 2
Concrete e/D x 10^4	1.25	1.25	1.25	1.25	1.25	1.25
Steel, use	Tab. 4	Tab. 4	Tab. 4	Tab. 4	Tab. 4	Tab. 4
Steel, e/D x 10^4	3.30	3.30	3.30	3.30	3.30	3.30
Tee Losses						
Ratio D_r/D=2/3, use		Tab. 5	Tab. 5	Tab. 5	Tab. 5	Tab. 5

Table 8 STABILITY STUDY

	MANUAL COMP.	Gate Curve No. 1 Test 1	Gate Curve No. 1 Test 2	Gate Curve No. 1 Test 3	Gate Curve No. 2	Gate Curve No. 3	Gate Curve No. 4
Initial Conditions							
Gate Opening %	79	79.22	68.57	68.57	68.57	68.57	68.57
Net Head on Turbine, ft.	120	120	130	130	130	130	130
Turbine Q_5, cfs	5960	5970	5500	5500	5500	5500	5500
Brake Horsepower	75,000	75,000	75,000	75,000	75,000	75,000	75,000
New Conditions							
Desired Brake Horsepower	78,000	78,000	78,000	78,000	78,000	78,000	78,000
Gate Opening %							
Reservoir Ws Elev., E_R	1540.0	1544.3	1553.8	1553.8	1553.8	1553.8	1553.8
Tailwater Elev.	1414.9	1420.0	1420.0	1420.0	1420.0	1420.0	1420.0
Elev. Surge Tank Base, E_s	1515.0	1515.0	1515.0	1515.0	1515.0	1515.0	1515.0
Elev. of Distributor, E_t	1422.0	1422.0	1422.0	1422.0	1422.0	1422.0	1422.0
Lengths in Feet							
Upstrm Conduit, Total L_1	3580	3820.0	3820.0	3820.0	3820.0	3820.0	3820.0
Upstrm Conduit, conc., L_c	1300	1292.0	1292.0	1292.0	1292.0	1292.0	1292.0
Upstrm Conduit, Steel, L_s	2280	2528.0	2528.0	2528.0	2528.0	2528.0	2528.0
Between Tanks, L_3	40.0	40.0	40.0	40.0	40.0	40.0	40.0
Dnstrm Tank to Turbine, L_5	66.0	66.0	66.0	66.0	66.0	66.0	66.0
Riser, R	81.0	81.0	81.0	81.0	81.0	81.0	81.0
Diameters in Feet							
Surge Tank, D_f	70.0	70.0	70.0	70.0	70.0	70.0	70.0
Riser, D_r	16.0	16.0	16.0	16.0	16.0	16.0	16.0
Orifices, D_o	12.33	12.33	12.33	12.33	12.33	12.33	12.33
Penstock, D	24.0	24.0	24.0	24.0	24.0	24.0	24.0
Governor Time, Seconds							
Basis of Computation							
Accel. Head in Riser	NC	NC	C	NC	C	C	C
Accel. Head in L_5	NC	C	C	C	C	C	C
Head Loss @ Tees	C	C	C	C	C	C	C
Head Loss @ Orifice	C	C	C	C	C	C	C
Friction in Riser, R	C	C	C	C	C	C	C
Loss, Tank to Riser	C	C	C	C	C	C	C
Friction Factor f, varying	C	C	C	C	C	C	C
Other Data							
Temperature, °F	38.0	38.0	38.0	38.0	38.0	38.0	38.0
$\nu \times 10^5$ (kin. viscosity)	1.71	1.73	1.73	1.73	1.73	1.73	1.73
Surge Tank, K_t	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Form Losses in L_1	Min.	Min.	Min.	Min.	Min.	Min.	Min.
Trashrack, K_{tr}	0.03	0.03	0.03	0.03	0.03	0.03	0.03
Entrance, K_e	0.03	0.03	0.03	0.03	0.03	0.03	0.03
Gate Slots, K_s	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Transitions, K_{ts}	0.03	0.03	0.03	0.03	0.03	0.03	0.03
Horiz. Bends, K_{hb}	0.02	0.02	0.02	0.02	0.02	0.02	0.02
Vert. Bends, K_{vb}	0.03	0.03	0.03	0.03	0.03	0.03	0.03
Other, K_{ot}	0.20	0.20	0.20	0.20	0.20	0.20	0.20
Vel. Head, K_v	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Total; ΣK	1.35	1.35	1.35	1.35	1.35	1.35	1.35
Orifice Ratio D_o/D_r	0.771	0.771	0.771	0.771	0.771	0.771	0.771
Orifice Coeff., C_o	0.70	0.70	0.70	0.70	0.70	0.70	0.70
Friction Losses							
Smooth-pipe, use	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1
Concrete, use	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1
Concrete e/D x 10^4							
Steel, use	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1	Tab. 1
Steel, e/D x 10^4							
Tee Losses							
Ratio D_r/D=2/3, use		Tab. 5	Tab. 5	Tab. 5	Tab. 5	Tab. 5	Tab. 5

Legend: C - Computed
NC - Not computed

ts for Full-Load Rejection

ith reference to Table 6, all cases are based upon rejecting full load on rbine, 142,000 horsepower output, at a maximum net head on the turbine 3 feet for steady state. For this condition the gate opening is restricted 3 per cent for all computer solutions (70 per cent for manual compu-s) in order to avoid overload on the generator, and closure of the gates hieved in 3.31 seconds (computer) at a uniform rate of 20 per cent per d. Turbine-performance values (Eq. (20)) have been obtained by linear polation in a table of the form indicated by Table 3. The actual turbine-rmance table that was used in machine computations gave Q_5 values for H at closer intervals (ΔG = 5 between 0 and 100, ΔH = 5 feet between d 240 feet).

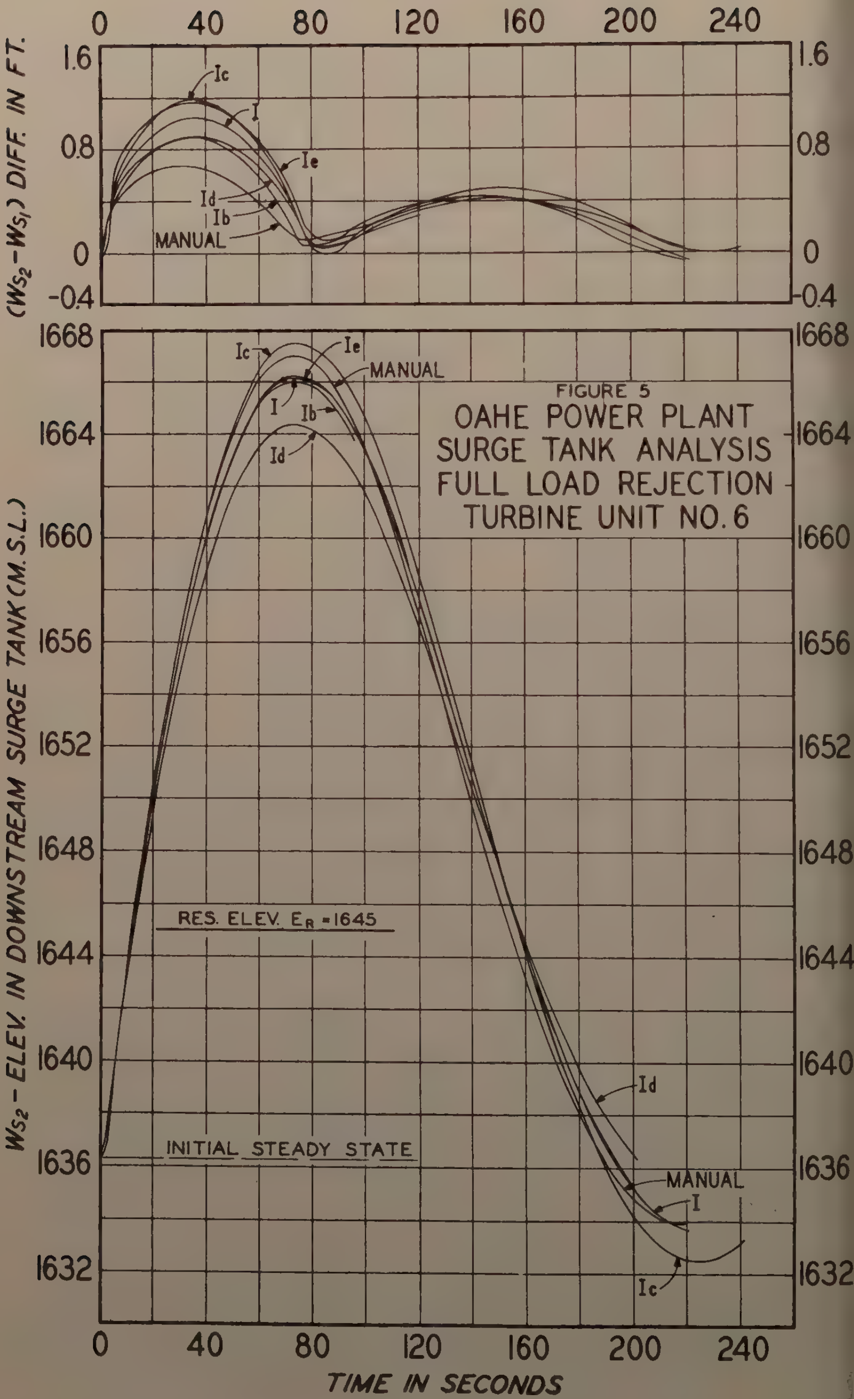
gs. 5 and 6 indicate some of the results that were obtained for the re-n cases. Fig. 5 depicts the water surface elevation of the downstream tank and the difference in water surface elevation between the two tanks function of time for all cases indicated in Table 6. Case I and Manual outations are based upon the same data and conditions, except that for the al computations L_1 is over 200 feet shorter, initial Q_5 and G are slight-gher, and acceleration heads in the risers and in the length L_5 are not outed. The basis of computation for all other cases shown on Figs. 5 and he same as for Case I, except that for:

ase Ib, acceleration heads in the risers have not been computed.
ase Ic, the orifice coefficient is 0.8 instead of 0.7.
ase Id, the orifice coefficient is 0.6 instead of 0.7.
ase Ie, the length L_3 is 80 feet instead of 40 feet, L_1 and L_5 being some-what smaller as a consequence.

amination of the curves on Fig. 5 indicates successful application of ine methods to rejection problems. It is noted that: (a) the numerical of the orifice coefficient has the most pronounced effect upon the water ce elevations in the tanks and (b) in comparison with the results of manu-mputations, the difference in water surface elevations between tanks is antially greater for all computer cases during the first quarter cycle. l machine-computation time for each of the cases shown on Fig. 5 aver-slightly over one hour. The Q values for Case Id on Fig. 6 are more or typical of those for all other computer cases indicated on Fig. 5.

ts for Full-Load Demand

ith reference to Table 7, all cases are based upon picking up load when eservoir elevation is at minimum design level (E_R = 1540). The rate of ng the wicket gates is linear in all cases, the rate being 20 per cent per d (governor time of 5 seconds) for all except IIa. For Case IIa, the nor time is 10 seconds. Applicability of Table 3 to load-demand cases same as for the load-rejection cases previously discussed. Case II and al Computations (Table 7) are based upon the same data and conditions t that for the latter: L_1 is over 200 feet shorter; initial steady-state G o (instead of 5.0); gate opening at end of governor action is 90 per cent ad of 100 per cent); tailwater elevation is 1420 (instead of 1421.7); and eration heads in the risers and in the length L_5 are not computed. All ter cases are based upon an initial steady-state discharge Q_5 of 390 (corresponding to initial G = 5.0) instead of zero. A comparatively small



$(W_{S_2}-W_{S_1})$ DIFF. IN FT.
Ic
I
Ie
Id
Ib
MANUAL
FIGURE 5
OAHE POWER PLANT
SURGE TANK ANALYSIS
FULL LOAD REJECTION
TURBINE UNIT NO. 6
RES. ELEV. E_R = 1645
INITIAL STEADY STATE
W_{S_2} - ELEV. IN DOWNSTREAM SURGE TANK (M.S.L.)
TIME IN SECONDS

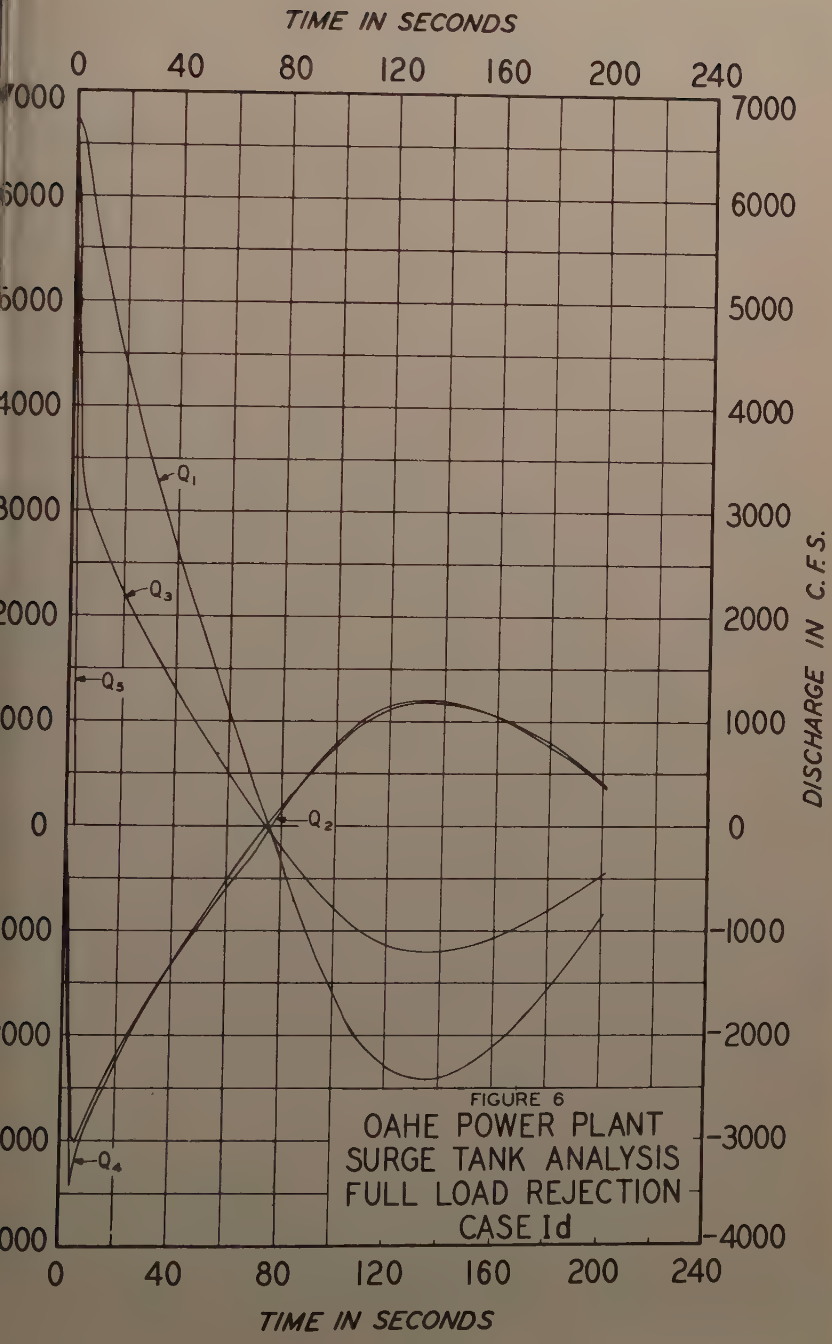


FIGURE 6
OAHE POWER PLANT
SURGE TANK ANALYSIS
FULL LOAD REJECTION
CASE Id

finite value of Q_5 at the start seems to be necessary because of the machine methods of solving differential equations that has been utilized. A smaller value than 390 c.f.s. might have been used successfully. However, provided that the initial Q_5 is small in comparison to the full-load Q_5, the over-all results are not likely to be significantly affected.

Figs. 7, 8, and 9 indicate some of the results that were obtained for the demand cases. All data shown under Case II in Table 7 also apply to Cases II-1 and II-2, except that the surge tank base elevation E_s of 1515 does not apply to II-2. Under Case II-1, the surge tank water surface drops down into the riser, but under Case II-2, E_s is sufficiently lower (about 3 feet) so that the water surface does not recede into the riser. (With this small difference the change in R is ignored.) The basis of computation for all other computer cases is the same as for Case II-2 except that for:

Case IIa, the governor time is 10 seconds instead of 5 seconds, and with E_s = 1515 the water surface drops down into the riser.

Case IIb, acceleration heads in the risers have not been computed.

Case IId, the orifice coefficient is 0.6 instead of 0.7.

Case IIe, L_3 is 80 feet instead of 40 feet thus causing L_1 and L_5 to be somewhat smaller, and K_e' is 1.00 (re-entry tube) instead of 0.5.

Comparison of manual and machine results on Fig. 7 demonstrates that machine methods have been applied successfully also to full-load demand problems. The question of whether or not to permit the water surface to drop down into the riser in actual operation is immaterial insofar as the purpose of this paper is concerned. Results are now available as a basis for decision in that respect. It will be noted from Figs. 7 and 8 that machine results indicate oscillations of rather large amplitude between tanks for a short period, when the water surface is permitted to recede into the risers. Cases II-1 and IIa are directly comparable except for governor time, and hence results for these two cases are plotted separately for comparison on Figs. 8 and 9. Doubling the governor time (IIa) seems to make little difference in over-all results for the Oahe surge tanks as such; that is, low point of water surface elevation in the risers (Fig. 8) is about the same and the timing therefor is also about the same. But, of course, with a slower governor, pickup of the turbine load is materially delayed, as comparison of Q_5 values on Fig. 9 indicates. It is interesting to note from this figure that except for a slight lag time, the results of Case IIa are almost a carbon copy of those of Case II-1. This demonstrates that the computer does the job with slavish adherence to instructions, and, as a consequence, the analyst cannot afford to slight or overlook any pertinent detail in the preparation of the machine program. Actual machine-computation time for each of the demand cases discussed above averaged about one and one-half hours.

Surge-Tank Stability Analyses

In normal practice, stability analyses are made for small power-demand changes at minimum design head on the turbine in the region of drooping efficiency (between gate opening for maximum efficiency and full gate.) Ideally what is desired is to go directly from the initial steady-state power Bhp_1 to a new steady-state power Bhp_2. For example, on Fig. 11 it is desired to proceed directly from the point M_1 (Bhp_1 = 75,000, H_1 = 120) to point M_2 (Bhp_2 = 78,000, H_2 = 119.6), the new steady state for constant turbine output

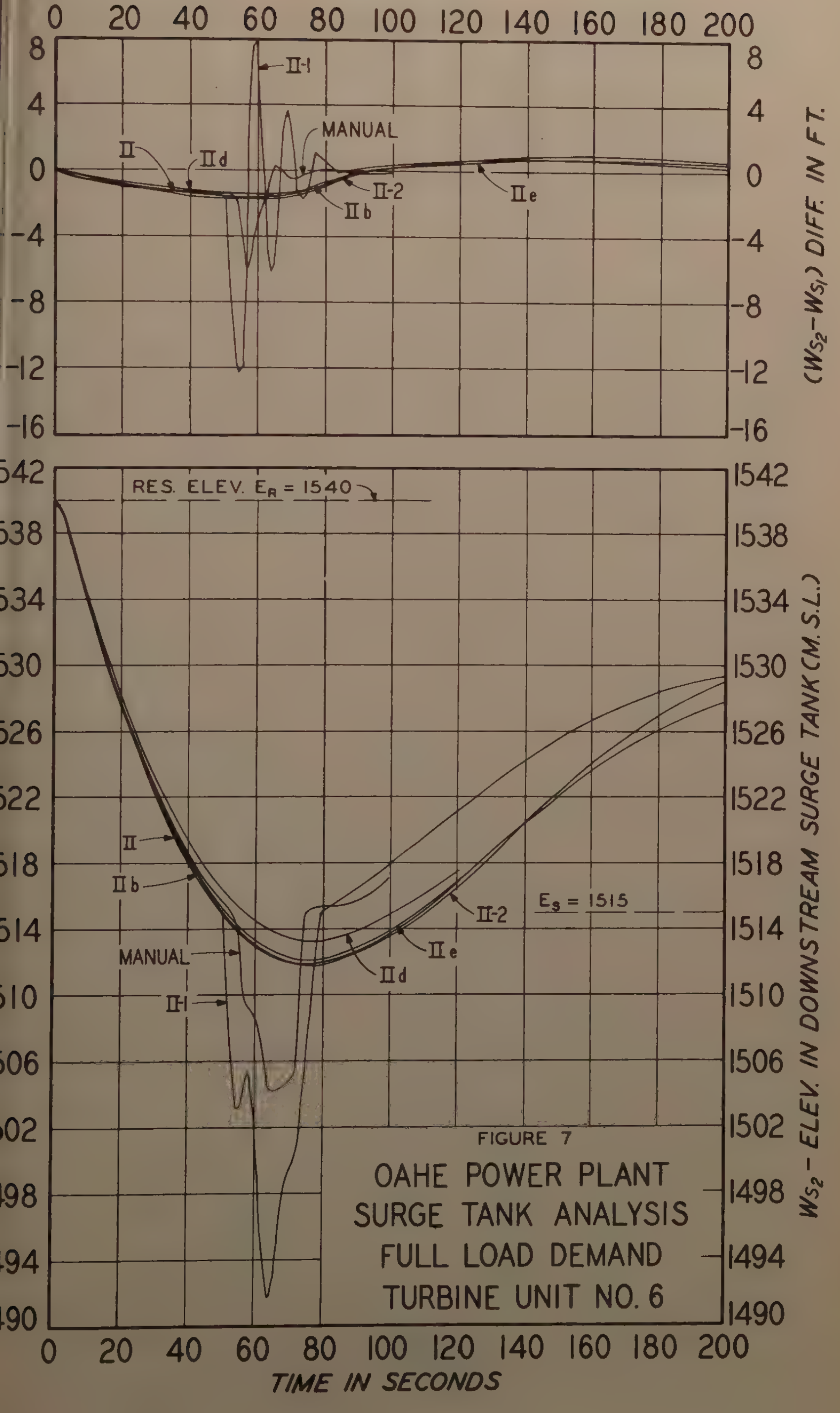


FIGURE 7
OAHE POWER PLANT
SURGE TANK ANALYSIS
FULL LOAD DEMAND
TURBINE UNIT NO. 6

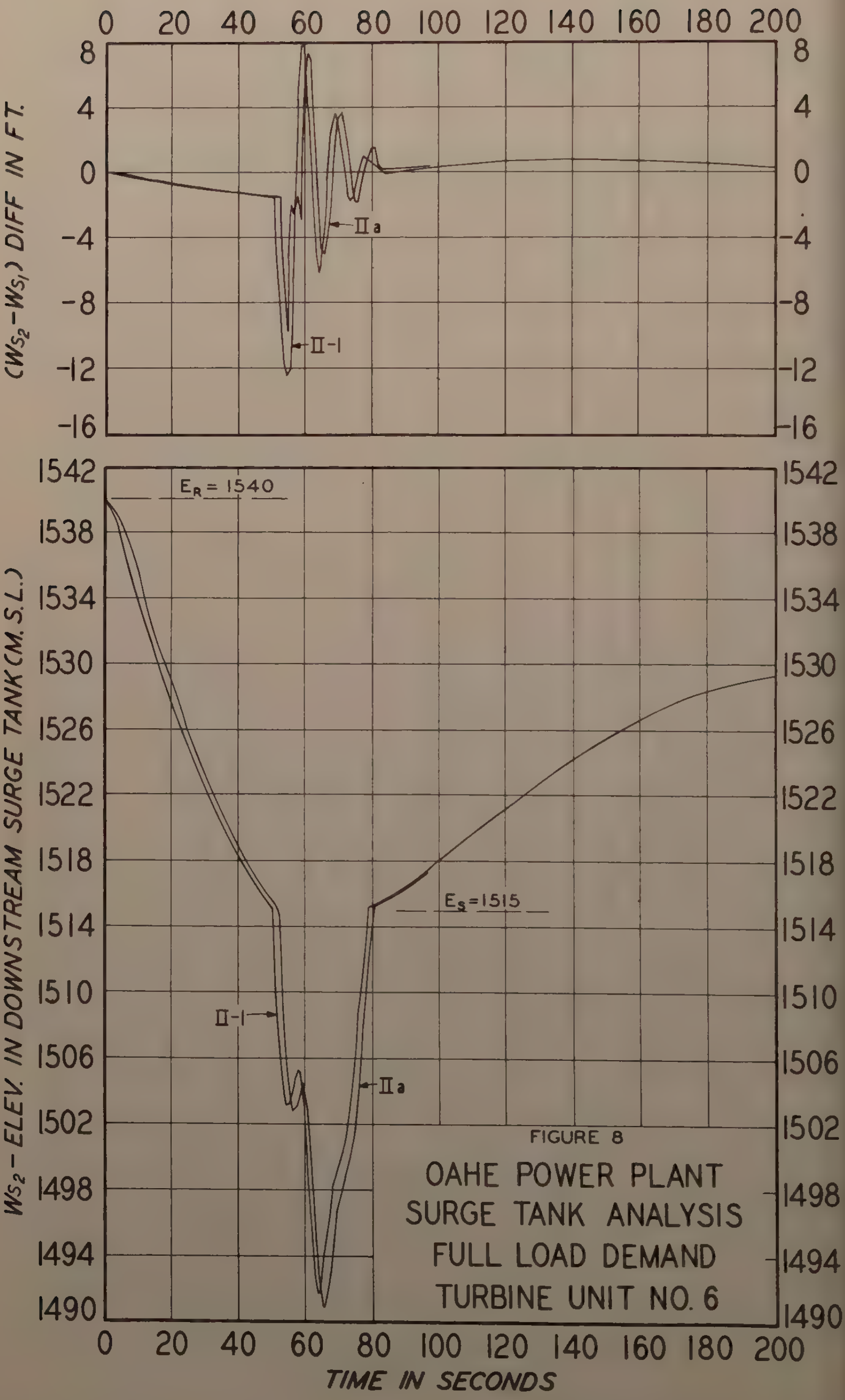
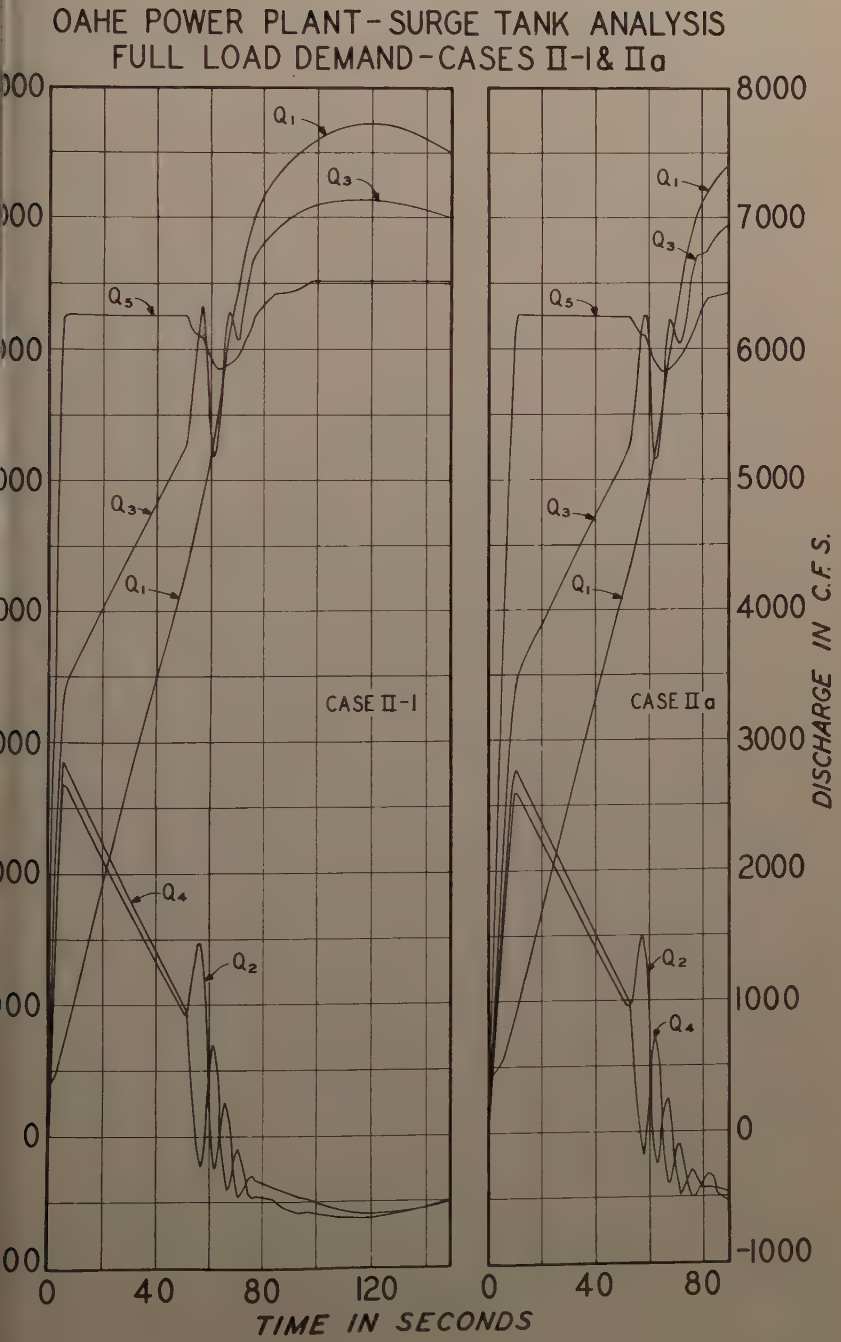


FIGURE 8

OAHE POWER PLANT
SURGE TANK ANALYSIS
FULL LOAD DEMAND
TURBINE UNIT NO. 6

FIGURE 9


OAHE POWER PLANT - SURGE TANK ANALYSIS
FULL LOAD DEMAND - CASES II-1 & IIa
Q1
Q3
Q5
Q4
Q2
CASE II-1
CASE IIa
8000
7000
6000
5000
4000
3000
2000
1000
0
-1000
0
40
80
120
0
40
80
TIME IN SECONDS
DISCHARGE IN C.F.S.

of 78,000 horsepower. However, this is not possible, for sudden gate movements in response to rapid governor action set up transients in the wh surge system. These transients will die out in a reasonable length of time the system is properly designed. By conventional methods, it is assumed purposes of analysis that once entry onto the curve Bhp_2 is made, at whate point that may be, the power delivered by the turbine will have a constant value Bhp_2 thereafter. The oscillations in Q_5, H, and G which occur with Bhp_2 constant are then determined by manual methods of arithmetic integration.(2) Such methods are necessarily based upon simplifying assumpti as a result of which the total transient problem is treated as one of separa multiple parts. In essence, the hydraulic system is divorced from the inte connected mechanical and electrical system for the practical purpose of an lyzing the hydraulic transients as such. Use of this same approach to solutions by computer (employing the method of solving differential equation previously discussed) has been unsuccessful, but only in the sense that the computer has failed to produce the same results as those obtained by manu computations. In another sense the computer study has been quite fruitful that it has forced a reappraisal of the total "transient" problem and has hi lighted the shortcomings of manual methods of analysis. Discussion of the need for comprehensive analysis of power plant transients as an integrable whole is presented in another section of the paper.

With reference to Table 8, all computer runs were made starting with Bhp_1 = 75,000 for initial steady state at a net head on the turbine of either or 130 feet (the former value being the minimum design head for the Oahe turbine.) The gate-time curves indicated in Table 8 are shown on Figs. 4 10, on which G_o is the initial steady-state gate opening (line one of Table Curve No. 1 on Fig. 4 represents part of the full-range gate-time curve furnished by the Oahe turbine manufacturer. Its relation to Curves No, 2, and 4 is indicated on Fig. 10. Curves 2, 3, and 4 are entirely arbitrary. for all computer runs are exactly the same in all respects except for indic ed initial H values (line 2 in Table 8), gate curves, and indicated inclusion exclusion of acceleration heads in the risers (line 20 in Table 8).

The first computer run that was made for stability analysis of surge ta has been designated Test No. 1 and involved use of Gate-Time Curve No. (Table 8). Data for this test are exactly the same as for Manual Computations, except for the inclusion of acceleration heads in L_5 and the differe in L_1. The path taken by the computer in trying to reach Bhp_2 = 78,000 is indicated by Test No. 1 on Fig. 11. Full gate was reached at 1.1 seconds fore Bhp_2 was reached. In following the paths indicated by Tests No. 1, 2 and 3 on Fig. 11, the computer was instructed to follow the gate-time curv given to it (in tabular form) until it reached the curve Bhp_2, after which c trol for development of the transient would revert to that curve. (That is, thereafter the function represented by Eq. (21) instead of Eq. (20) would h to be satisfied as well as other relationships.) Also, the computer was pr grammed to stop whenever it reached full gate, since oscillations past ful gate are unacceptable.

Test No. 1 having failed, Test No. 2 was started from a region (H = 13 where the efficiency curve is flat and turbine efficiency is practical const down to H = 120. Again, the computer followed a devious path, reaching E at 1.36 seconds (Fig. 11), after which it continued along the curve Bhp_2 to gate. With the thought that, perhaps, riser acceleration effects caused th devious path for Test No. 2, Test No. 3 was started from the same point

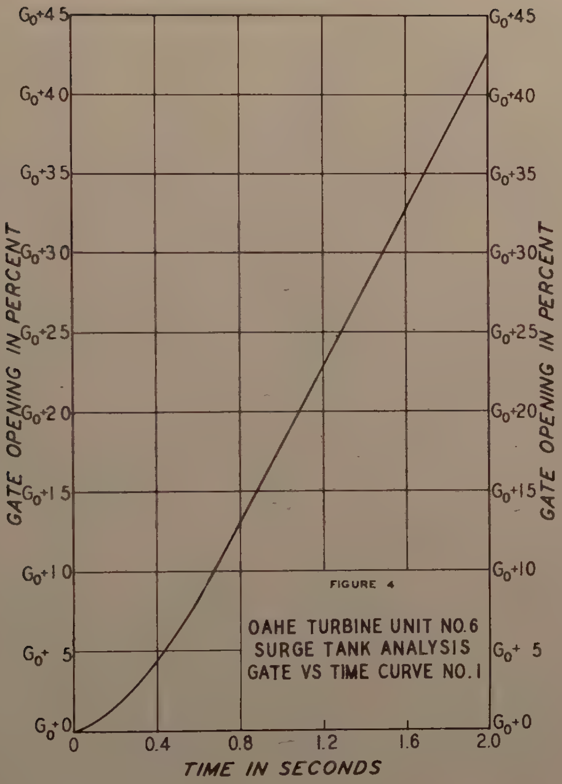


FIGURE 4

OAHE TURBINE UNIT NO.6
SURGE TANK ANALYSIS
GATE VS TIME CURVE NO.1

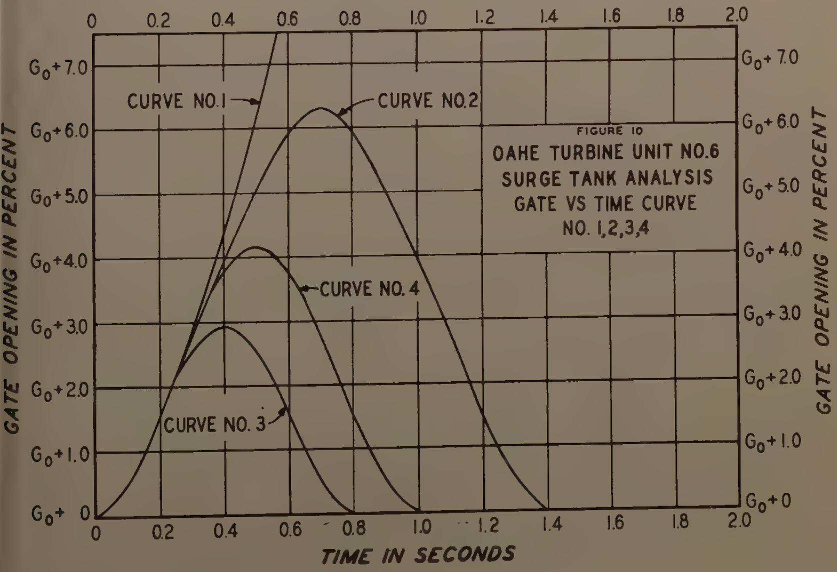


FIGURE 10

OAHE TURBINE UNIT NO.6
SURGE TANK ANALYSIS
GATE VS TIME CURVE
NO. 1,2,3,4

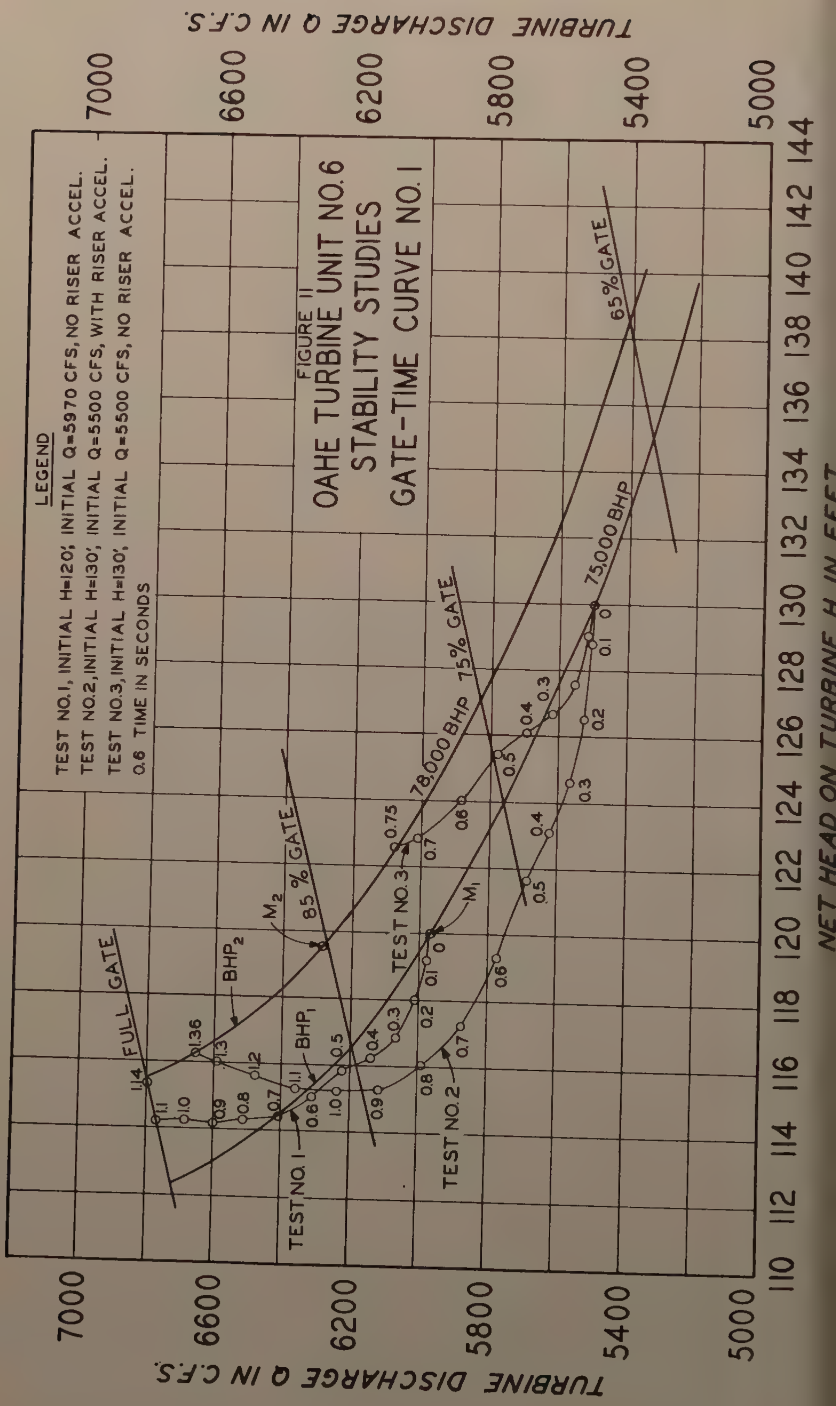


FIGURE 11
OAHE TURBINE UNIT NO. 6
STABILITY STUDIES
GATE-TIME CURVE NO. 1

130) with the effects of riser acceleration excluded. This time, the path wed by the computer was satisfactory until the Bhp_2 curve was reached 75 seconds (Fig. 11), but after that the computer followed the Bhp_2 curve rd to full gate once again. These failures to establish a transient (simi- o that obtained by Manual Computations) that would produce oscillations , G, and H along the constant Bhp curve of 78,000 without at any time hing full gate do not indicate that the Oahe surge system is unstable in egion investigated. Rather, they indicate that any transient set up in the ner of conventional methods of analysis is likely to resemble the actual sient only by accident, for these methods ignore (with justification*) the llations of the gates with time due to other effects at the turbine and rnor end of the surge system. In anticipation of any doubts that may arise erning the adequacy of the machine program, sufficient data are given in paper for the reader to assure himself that the computer has done the job cordance with instructions. Machine print-out of results for Test No. 2 given in the upper part of Table 9. These data together with the equations ppendix III, data given in Tables 1, 5, and 8, and gate-time Curve No. 1 on 4 will permit checks on machine arithmetic in a relatively short time. ll be noted that the machine does not print out values of h_{a_2} and h_{a_4}; they be obtained by separate computation.

hen a load change is called for by external electrical power require- s, oscillations are set up in the turbine governor system itself during the several seconds. By virtue of interconnection, the gates also oscillate. effort to represent at least the first one-half cycle of these oscillations, -time Curves No. 2, 3, and 4 (Fig. 10) were invented, using the "dead" "cushion" ends of the manufacturer's curve (of which Curve No. 1 is a to form the peak regions and the tail-ends. Application of these gate- curves then gave the results shown on Fig. 12. In all three cases the puter started and stayed within a region in which turbine efficiency is tically constant. In following the path indicated by Curve No. 2 on Fig. 12, omputer reached Bhp_2 at 0.8 second, after which it proceeded along Bhp_2 ll gate in the same manner as before. (Machine print-out of some of the lts based on use of Curve No. 2 is given in the lower half of Table 9). At point in the study a program change was made by which the machine was ucted to ignore the first intersection with Bhp_2 and to follow gate-time e No. 2 (Fig. 10) until it reached Bhp_2 a second time, after which control d revert to Bhp_2. For this condition the machine stopped at 1.2 seconds the end of gate-time Curve No. 2 shown on Fig. 10), and this test was doned. This same procedure was applied to gate-time Curves No. 3 and h the results shown on Fig. 12. In following the path marked Curve No. 3, omputer followed instructions perfectly, reaching Bhp_2 a second time at second, after which it followed Bhp_2 downward to the right and stopped 125 seconds. It stopped at this point because it had reached the end of able giving H as a function of Q_5 along the curve Bhp_2, the end of the table g been purposely set at a limit of H = 140. Gate-time Curve No. 4 was ed with the aim of having the path loop back and intersect Bhp_2 in the

ification exists because, normally, these oscillations cannot be described quately without bringing into the problem all the relationships that tie the raulic, mechanical and electrical systems together. Manual analyses on basis would be totally impracticable.

TABLE 9 OAHE POWER PLANT SURGE TANK ANALYSIS - - STABILITY STUDY - - TURBINE UNIT NO. 6

t	Q_1	h_{L_1}	h_{a_1}	P_1	$T_{1,7}$	$T_{1,2}$	Ws_1	h_{L_2}	P_8	Q_3	h_{L_3}	h_{a_3}	Q_4	Ws_2	h_{L_4}	$T_{8,9}$	$T_{8,14}$	$V_5^2/2g$	h_{L_5}	h_{a_5}	H	G	Q_5
Gate Curve No. 1		Tailwater Elevation 1420.00							Reservoir Elevation E_R 1553.761														
0	5500	5.902	0.000		0.000	0.000	1547.86	0.000		5500	0.029	0.000	0	1547.83	0.000	0.000	0.000	2.299	0.048	0.000	130.080	68.57	5500
0.10	5500	5.902	0.585	1547.27	0.006	0.001	1547.86	0.000	1547.09	5503	0.029	0.150	3	1547.83	0.000	0.001	0.005	2.303	0.048	0.500	128.838	69.07	5505
0.20	5501	5.903	1.676	1546.18	0.026	0.004	1547.86	0.000	1545.72	5513	0.029	0.402	14	1547.83	0.001	0.004	0.029	2.321	0.049	1.410	126.558	70.07	5527
0.30	5501	5.905	2.576	1545.28	0.064	0.009	1547.86	0.003	1544.58	5532	0.030	0.610	35	1547.83	0.004	0.010	0.075	2.355	0.049	2.171	124.637	71.47	5567
0.40	5502	5.908	3.263	1544.59	0.115	0.016	1547.86	0.008	1543.68	5557	0.030	0.768	65	1547.83	0.012	0.019	0.138	2.402	0.050	2.757	123.134	72.97	5622
0.50	5504	5.911	3.916	1543.93	0.178	0.025	1547.86	0.020	1542.81	5588	0.030	0.918	101	1547.83	0.029	0.031	0.219	2.460	0.051	3.317	121.681	74.67	5689
0.60	5506	5.914	4.981	1542.87	0.257	0.036	1547.85	0.041	1541.42	5626	0.031	1.161	147	1547.82	0.061	0.045	0.322	2.532	0.053	4.213	119.362	76.87	5773
0.70	5508	5.919	5.927	1541.92	0.355	0.050	1547.85	0.077	1540.16	5672	0.031	1.374	203	1547.82	0.116	0.063	0.452	2.623	0.055	5.007	117.265	79.27	5875
0.80	5510	5.924	6.041	1541.40	0.467	0.065	1547.84	0.130	1539.41	5725	0.032	1.484	266	1547.81	0.200	0.085	0.606	2.727	0.056	5.432	116.048	81.67	5991
0.90	5513	5.929	6.730	1541.10	0.587	0.082	1547.84	0.201	1538.95	5780	0.032	1.538	333	1547.80	0.314	0.108	0.775	2.840	0.059	5.655	115.298	84.07	6113
1.00	5515	5.935	6.686	1541.14	0.710	0.100	1547.83	0.290	1538.89	5836	0.033	1.512	403	1547.79	0.457	0.134	0.954	2.957	0.061	5.588	115.240	86.47	6238
1.10	5518	5.940	6.606	1541.21	0.833	0.117	1547.82	0.391	1538.87	5890	0.033	1.474	470	1547.78	0.624	0.159	1.136	3.074	0.063	5.474	115.275	88.94	6360
1.20	5520	5.946	6.384	1541.43	0.954	0.134	1547.81	0.503	1539.04	5943	0.034	1.401	536	1547.77	0.811	0.185	1.320	3.190	0.065	5.229	115.619	91.41	6479
1.30	5523	5.951	6.140	1541.67	1.069	0.150	1547.80	0.622	1539.25	5992	0.034	1.321	599	1547.75	1.012	0.210	1.500	3.301	0.067	4.955	116.025	93.88	6591
1.36	5524	5.950	6,000	1541.81	1.138	0.159	1547.79	0.698	1539.37	6022	0.035	1.272	637	1547.74	1.143	0.226	1.611	3.369	0.068	4.786	116.269	95.43	6658
1.44	5526	5.954	6.211	1541.61	1.222	0.171	1547.78	0.795	1539.06	6057	0.035	1.297	682	1547.73	1.310	0.245	1.738	3.450	0.070	4.880	115.821	97.94	6738
1.50	5527	5.957	6.374	1541.45	1.287	0.180	1547.77	0.875	1538.81	6084	0.035	1.318	717	1547.72	1.449	0.261	1.820	3.515	0.071	4.958	115.471	100.00	6801
Gate Curve No 2		Tailwater Elevation 1420.00							Reservoir Elevation E_R 1553.761														
0	5500	5.902	0.000		0.000	0.000	1547.86	0.000		5500	0.029	0.000	0	1547.83	0.000	0.000	0.000	2.299	0.048	0.000	130.080	68.57	5500
0.10	5500	5.902	0.678	1547.18	0.005	0.001	1547.86	0.000	1546.98	5503	0.029	0.171	2	1547.83	0.000	0.001	0.005	2.303	0.048	0.577	128.648	69.12	5505
0.20	5501	5.903	1.704	1546.15	0.026	0.004	1547.86	0.000	1545.70	5513	0.029	0.408	14	1547.83	0.001	0.004	0.029	2.321	0.049	1.434	126.500	70.09	5527
0.30	5501	5.905	2.421	1545.43	0.062	0.009	1547.86	0.002	1544.75	5531	0.030	0.574	34	1547.83	0.003	0.010	0.073	2.354	0.049	2.042	124.958	71.32	5566
0.40	5502	5.907	2.836	1545.02	0.109	0.015	1547.86	0.008	1544.21	5554	0.030	0.670	61	1547.83	0.011	0.018	0.130	2.396	0.050	2.403	124.022	72.51	5615
0.50	5503	5.910	2.963	1544.89	0.160	0.022	1547.86	0.016	1544.00	5579	0.030	0.699	91	1547.83	0.023	0.027	0.195	2.443	0.051	2.525	123.671	73.57	5670
0.60	5505	5.912	2.870	1544.98	0.212	0.029	1547.85	0.028	1544.06	5604	0.030	0.677	121	1547.82	0.041	0.037	0.263	2.491	0.052	2.462	123.773	74.47	5725
0.70	5506	5.914	2.274	1545.57	0.260	0.036	1547.85	0.042	1544.75	5627	0.031	0.538	149	1547.82	0.062	0.046	0.326	2.535	0.053	1.979	124.923	74.87	5776
0.80	5506	5.916	1.039	1546.81	0.290	0.041	1547.85	0.052	1546.23	5642	0.031	0.252	167	1547.82	0.079	0.052	0.370	2.564	0.053	0.964	127.411	74.47	5809
0.81	5506	5.916	0.948	1546.90	0.292	0.041	1547.85	0.052	1546.34	5642	0.031	0.231	168	1547.81	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

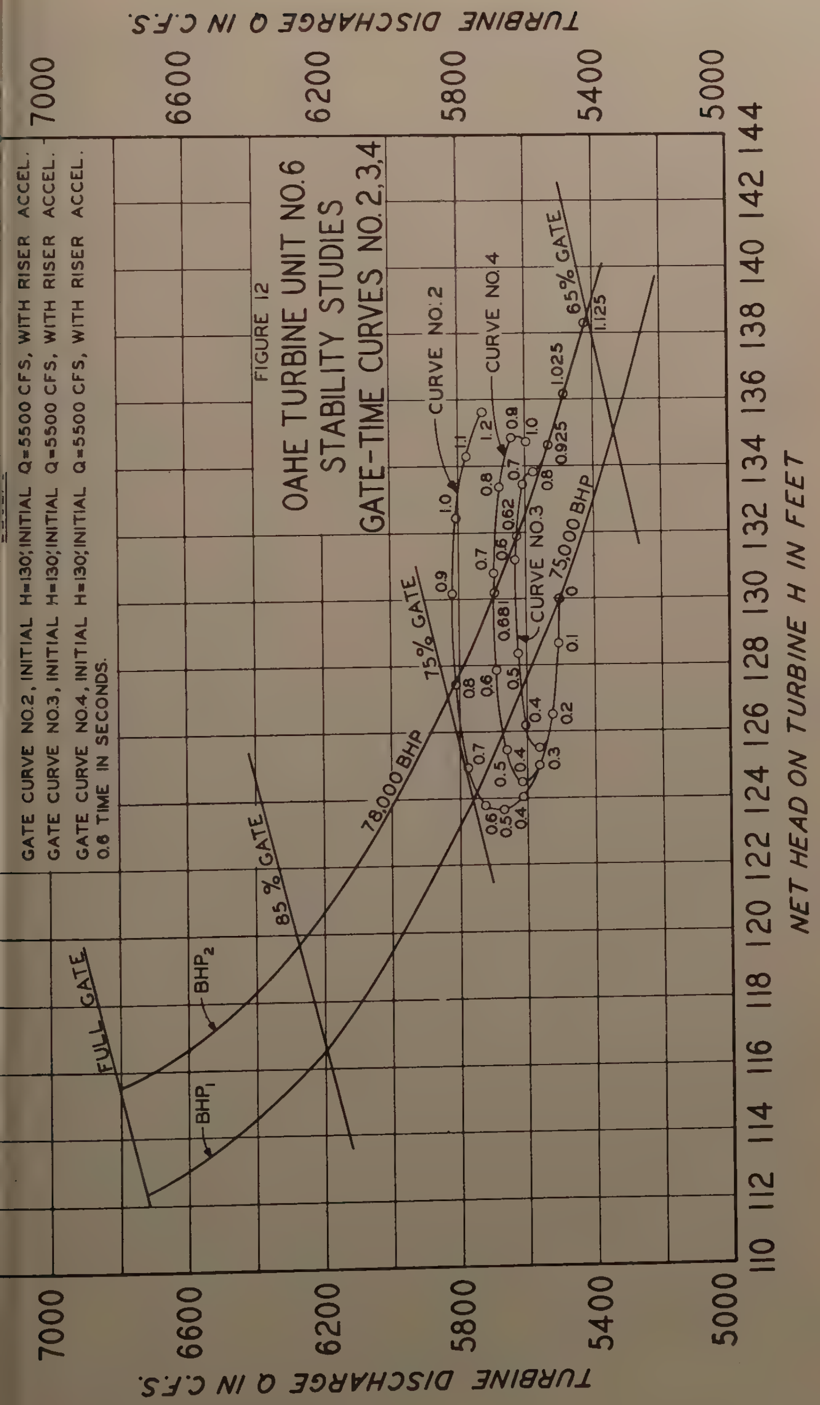


FIGURE 12
OAHE TURBINE UNIT NO.6
STABILITY STUDIES
GATE-TIME CURVES NO.2,3,4
GATE CURVE NO.2, INITIAL H=130; INITIAL Q=5500 CFS, WITH RISER ACCEL.
GATE CURVE NO.3, INITIAL H=130; INITIAL Q=5500 CFS, WITH RISER ACCEL.
GATE CURVE NO.4, INITIAL H=130; INITIAL Q=5500 CFS, WITH RISER ACCEL.
0.6 TIME IN SECONDS.
FULL GATE
85 % GATE
75% GATE
65% GATE
BHP_1
BHP_2
78,000 BHP
75,000 BHP
CURVE NO.2
CURVE NO.3
CURVE NO.4
TURBINE DISCHARGE Q IN C.F.S.
NET HEAD ON TURBINE H IN FEET
7000
6600
6200
5800
5400
5000
110 112 114 116 118 120 122 124 126 128 130 132 134 136 138 140 142 144

opposite direction from that which had been indicated by Curve No. 3. The computer started a return loop along Curve No. 4 as indicated by the points at 0.9 and 1.0 second on Fig. 12 and then stopped at 1.0 second (the end of gate-time Curve No. 4 on Fig. 10). At this point it was decided that there w altogether too much arbitrary judgment being exercised, and further compu er studies of stability were not undertaken.

Machine-computation time for all these tests was comparatively small. However, a good deal of time was spent in analyzing machine results and in trying to establish a transient using a function of the form given by Eq. (21) as a control. It is concluded from the experience thus gained that analyses utilizing this procedure yield artificial results and that, as a consequence, the results of conventional methods of analysis of surge-tank stability along the lines previously described are incomplete and inconclusive. This does not raise or imply any reflection on the excellent work that has been done in connection with hydraulic design of power plants. For, it has long been rec nized that simplifications are a practical necessity in order to render powe plant "transient" problems tractable for solution by manual methods in a reasonable length of time. However, successful adaptation for computer so lutions of very complex problems in many fields of engineering and science now raises the question as to the necessity for continuing to use simplified procedures that yield results that are not entirely satisfactory.

Some Ideas for Comprehensive Study by Digital Computer

The ideas presented in this section of the paper have been extracted fro an unpublished "Proposal for Comprehensive Study by Digital Computer of Hydraulic and Governing Transients Affecting Design and Operation of a Power Plant System", prepared by the writer on his own initiative and submitted to the Corps of Engineers for consideration. This proposal includes the derivation of necessary equations and suggested methods of machine an sis and computation for undertaking a comprehensive study, which is only briefly discussed below because of space limitations. The only reason for viting attention to the proposed study at this time is to indicate to the engineering profession that efforts are being made to overcome the limitations conventional methods.

Formidable limitations of time and means have required up to now that power plant system be treated as a divisible whole for purposes of analysis of hydraulic and governing transients induced by external power-load variations. (As used herein, the term "power plant system" includes the surge system as previously defined and one power unit, turbine and generator, su plying load on an independent basis.) With the advent of powerful, high-spe computers, it appears that, for the first time, it is practicable to analyze t total "transient" problem as a "feedback" system of interwoven effects rat than as a problem having separable, multiple parts. Studies of water hamn surge-tank effects, turbine-speed regulation, and governing stability need longer be made on an essentially separate basis if a computer program car be developed to handle the total problem as such. Recent analog-computer studies by Koenig and Knudtson[9] are only a step toward the ultimate goal, for these studies involve simplifications which tend to cast some doubt on t validity of results obtained thereby.

appears that studies of the "total problem" can be made with conclusive ilts only when the relationships controlling hydraulic behavior in the uit, penstock, and surge-tank system are solved simultaneously with the iecting relationships between turbine output, the governor, machine in- a and speed, and external power load. To do so requires the simultaneous tion of a great number of equations, short time step by short time step. time step would probably have to be in the order of hundredths of a nd (or less in some cases) in order to account for all pressure dis- ances and reflections* generated during the existence of a transient. ously, such an approach is totally impractical, if not impossible, for ual computation, but, apparently, not for a modern digital computer hav- he capability and computational speed comparable with those of an IBM computer.

has been roughly estimated by the writer that the machine time required total solution on such a computer would be well within practical range, if iecessary machine program can be successfully developed. A total so- n would yield time histories of velocity and piezometric heads at a large ber of pre-determined points within the surge system as well as time ories of turbine power output, turbine discharge and head, turbine and rator speed for any assumed or given external power-load variation with applied at the power unit under study. Important quantities such as e tank size, orifice characteristics, penstock and conduit characteristics, hine inertia, governor characteristics, etc., would all be introduced in otal problem as parameters. By the proposed methods, water-hammer mass-oscillation phenomena would not be treated as separate entities. e from the basic assumption of essentially independent operation of the er unit, no other simplifications of important significance appear to be ssary for the methods of solution that have been proposed. To accomplish his on a digital computer requires the simultaneous solution by numerical iods of partial-differential equations within the surge system and of ordi- differential equations at the turbine end of the system. Methods for do- his are apparently available. It remains to be determined whether or not undamental partial-differential equations describing hydraulic behavior in the surge system can be solved by computer on a practical basis. The er believes that this can be done, but a machine programming job of great e will have to be performed before success is achieved.

CONCLUSIONS

machine program for the IBM 650 computer is now available for solving -rejection and full-load demand problems of a surge system comparable at at the Oahe project in one to two hours of machine-computation time case. The machine time per solution for installations having only one connected to the main conduit will be materially less. The program de- bed herein may be easily adapted for such cases. The relatively small required per machine solution and the comparatively small costs in- ed make it feasible to enlarge the scope of design studies of surge ems well beyond the limits ordinarily imposed by manual methods of so- n. By examining the results of a number of machine solutions readily

---

he study that is envisioned, effects of the compressibility of water and the ticity of flow boundaries would be included.

obtained as a result of changing one or two major factors from one machine run to another, a better understanding of problem phenomena for a particula installation will be obtained. As a consequence, construction savings may b achieved in some cases.

A digital computer program which will yield solutions for transient effec resulting from small-load changes from one steady-state power output to an other is not yet available despite considerable efforts to develop such a program. It may be entirely possible to develop such a program by utilizing methods of solution other than those described herein. However, it appears that any method which ignores the applicable relationships between turbine, governor, machine speed and inertia, and external power load for the purpo of analyzing the hydraulic transients as such is bound to yield inconclusive results. The premise that such simplifications give conservative results in sofar as surge-tank stability is concerned may be true but the degree of con servatism is not known in application to particular cases. For a long time need has existed for analyzing the total "transient" problem without the maj simplifications that are ordinarily made. When only manual methods of ana sis and computation were available, fulfillment of this need on a practical basis was not possible. With the advent of powerful, high-speed computers the prospects are now excellent that this need can be wholly satisfied. Som efforts have already been made toward the ultimate goal of adapting the tota "transient" problem of a power plant for solution by a digital computer in su a way that machine results for water hammer, surge-tank effects, turbine-speed regulation, and governing stability would be delivered in one package a few hours of machine-computation time. If successful adaptation is ultimately achieved, many benefits and possible construction savings will accru

ACKNOWLEDGMENTS

The permission granted by the Chief of Engineers, United States Army Corps of Engineers, to use the Oahe data and study results for the purposes of this paper is appreciated. The results of manual computations used for comparison in the paper were obtained from a design report prepared by Sverdrup and Parcel, Inc., in partial fulfillment of that firm's contract with the Omaha District for design of the Oahe power plant and surge tanks.

The excellent cooperation of the Omaha District and, particularly, of Messrs. Harris Engh and Marlin Bopp of that office is gratefully acknowledged. Special appreciation is expressed for the valuable programmi contributions of Messrs. Otto Steiner and William Gerkin, mathematicians the Missouri River Division office and for miscellaneous services rendere by Mr. Robert Burns, hydraulic engineer, also of that office.

APPENDIX I

Notation

A - Cross-sectional area in sq. ft. of conduit and penstock.

A_o - Area of orifice in sq. ft.

A_r - Area of riser in sq. ft.

- Area of surge tank in sq. ft.

hp - Brake horsepower (output of turbine)

o - Coefficient of orifice in $Q = C_o A_o \sqrt{2gh_o}$

- Coefficient for risers, $\frac{1}{2ga_r^2}$ or $\frac{1+K_e'}{2gA_r^2}$

D, C_D' - Coefficients of loss for diverging flow in tees, function of discharge ratio $(Q_{riser}/Q_{conduit})$

c, C_c' - Coefficients of loss for converging flow in tees, function of discharge ratio $(Q_{riser}/Q_{conduit})$

- General coefficient of loss for the tees in $T = C_t \frac{V^2}{2g}$

- Diameter in feet of conduit and penstock.

- Diameter of riser in feet.

o - Diameter of orifice in feet.

- Diameter of surge tank in feet.

- Absolute roughness length in feet for concrete.

- Absolute roughness length in feet for steel.

- Pre-assigned margin of relative error.

R - Elevation of reservoir in feet above M.S.L.

s - Elevation of base of surge tanks in feet above M.S.L.

- Elevation of centerline of turbine distributor.

- Darcy-Weisbach friction factor, function of Reynolds number.

, f_s - Friction factors in upstream conduit.

, f_4 - Friction factors in upstream and downstream risers, respectively.

- Friction factor in penstock between risers (Length L_3).

- Friction factor in penstock (Length L_5).

, F_s - Functions of Reynolds Number for concrete and steel.

F_g, F_q - Functions defined by Eq. (III-46), (III-45), and (III-49).

- Acceleration due to gravity in ft./sec.^{2}

- Gate opening in per cent for turbine wicket gates.

- Initial gate opening for steady state.

- Surface resistance loss in feet.

- Form loss in feet.

- Head loss in feet (may include all types of losses and velocity head).

h_{L1} - Head loss in feet in upstream conduit in length L_1 (includes all form and surface-resistance losses as well as velocity head), defined by Eq. (III-21).

h_{L2}, h_{L4} - Head losses in upstream and downstream risers, respectivel defined by Eqs. (III-26) and (III-29).

h_{L3} - Head loss in penstock L_3, defined by Eq. (III-24).

h_{L5} - Head loss in penstock length L_5, defined by Eq. (III-25).

H - Net head on turbine in feet.

h_o - Head on orifice in feet (piezometric drop).

h_a - Acceleration head in feet, defined as $\frac{L}{gA}\frac{dQ}{dt}$

h_{a1} - Acceleration head in upstream conduit (L_1).

h_{a2}, h_{a4} - Acceleration heads in upstream and downstream risers, respectively.

h_{a3} - Acceleration head in penstock length L_3.

h_{a5} - Acceleration head in penstock length L_5.

h_v - Velocity head in feet.

K - A constant, in general.

K_1 - A constant defined by Eq. (III-22).

K'_1 - A constant, $\frac{1}{2gDA^2}$

K_2, K_4 - Constants which apply to upstream and downstream risers, spectively, defined by Eqs. (III-28) and (III-31).

K_e - Entrance loss coefficient for upstream conduit.

K'_e - Entrance loss coefficient, surge tank to riser.

K_{tr} - Loss coefficient for trashracks.

K_s - Loss coefficient for all gate slots.

K_{ts} - Loss coefficient for all transitions.

K_{hb} - Loss coefficient for all horizontal bends.

K_{vb} - Loss coefficient for all vertical bends.

K_{ot} - Coefficient to account for all other form losses.

K_v - Velocity-head coefficient = 1.0.

L_1, L_3, L_5 - Lengths in feet in main conduit (see Fig. 1).

L_c - Portion of L_1 lined with concrete.

L_s - Portion of L_1 lined with steel.

N_R - Reynolds Number, $\frac{VD}{\nu}$

P	- Piezometric head in feet above M.S.L.
$P_1, P_2, P_3 \ldots$	- Piezometric heads at indicated points (see Fig. 1).
Q_1	- Discharge in c.f.s. in upstream conduit.
Q_2, Q_4	- Discharges in c.f.s. in upstream and downstream risers, respectively.
Q_3	- Discharge in c.f.s. in penstock length L_3 (Fig. 1).
Q_5	- Discharge in c.f.s. in penstock length L_5 (Fig. 1).
$\lvert Q \rvert$	- Indicates absolute value of Q (ignore sign).
R	- Height of riser in feet (Fig. 1).
R_y	- Relative error of y variables.
r_1, r_3, r_5	- Constants defined by Eqs. (III-1), (III-2), and (III-3).
r_2, r_4	- Constants defined by Eqs. (III-4) and (III-5).
t	- Time in seconds.
Δt	- Time interval.
T	- Tee loss in feet.
$T_{1,2}$	- Tee loss in feet between points 1 and 2 (Fig. 1).
$T_{1,7}, T_{8,9}, T_{8,14}$	- Tee losses in feet between indicated points.
Tw	- Tailwater elevation in feet above M.S.L.
V_1	- Velocity in ft./sec. in length L_1.
V_2, V_4	- Velocities in ft./sec. in upstream and downstream risers, respectively.
V_3	- Velocity in ft./sec. in length L_3.
V_5	- Velocity in ft./sec. in length L_5.
y	- Used to represent any one of variables, Q_1, Q_3, Q_5, Ws_1, and Ws_2.
Ws_1	- Water surface elevation (M.S.L.) of upstream surge tank.
Ws_2	- Water surface elevation (M.S.L.) of downstream surge tank.
ν	- Kinematic viscosity of water in feet squared per second.
α, β	- Functions defined by Eqs. (III-15) and (III-16).
ϕ, ψ	- Functions defined by Eqs. (III-18) and (III-19).

APPENDIX II

REFERENCES

1. Jaeger, C., "Engineering Fluid Mechanics", Blackie &Son, Ltd., London, 1956.
2. Rich, G. R., "Hydraulic Transients", Engineering Societies Monograph, McGraw-Hill Book Co., 1951.
3. Rouse, H., "Engineering Hydraulics", John Wiley & Sons, Inc., 1950.
4. Vogel, G., "Experiments To Determine The Loss In The Right Angle Pipe Tees", Translation from German, Tech. Memo. 299, U. S. Bureau of Reclamation, August 1932.
5. Johnson, W. E., "Missouri River Basin Plan In Operation", Proc. Paper 800, ASCE, Sept. 1955.
6. Pafford, R. J., Jr., "Operation Of Missouri River Main Stem Reservoirs", Proc. Paper 1370, ASCE, Sept. 1957.
7. Bradley, J. N. and Thompson, L. R., "Friction Factors For Large Conduits Flowing Full", U. S. Bureau of Reclamation, Eng. Monographs No. 7, March 1951.
8. Paynter, H. M., "Surge And Water Hammer Problems", Symposium, Transactions ASCE, Vol. 118, page 962, 1953.
9. Koenig, E. C. and Knudtson, H. A., "Computer Studies of Penstock and Governor Systems", Proc. Paper 1489, ASCE, December 1957.
10. Fritz, W. B. and Moraff, N., "Automatic Error Control In The Solution Of Differential Equations On The IBM 650", paper presented at ACM Meeting at U. of Houston, June 19, 1957.

APPENDIX III

Mathematical Relationships

Notation

Detailed notation is given in Appendix I. In general, the following notation is utilized, based upon the pound-foot-second system:

A	- Cross-sectional area	H	- Net head on turbine
C	- Coefficient (riser, orifice, tee)	K	- Coefficient for losses
D	- Diameter	L	- Length in conduit
E	- Elevation (M.S.L.)	N_R	- Reynolds Number
ε	- Absolute roughness length	P	- Piezometric head (M.S.L.)
F	- Function	Q	- Discharge
f	- Friction factor	R	- Height of riser
G	- Gate opening in per cent	T	- Tee loss

a	- Acceleration head	t	- Time
L	- Lost head	V	- Velocity
v	- Velocity head	Ws	- Water surface elevation

. Convention

he net head on the turbine and all piezometric heads are always positive. signs of all lost heads and acceleration heads are automatically assigned he equations that define them. The signs of all velocities will be the same hose of the corresponding discharges, since Q = AV. The sign convention all discharges is as follows:

All flows Q directed toward the turbine are given a positive (+) sign.
All flows Q directed away from the turbine are given a negative (-) sign.
Flows Q in the risers are positive when directed downward and negative when directed upward into the surge tanks.

hematical Relationships

The physical equations and functions required to solve surge problems for system shown in Fig. 1 are given in this appendix. The major variables ch vary with time t are: all Q's, all V's, all P's, all f's, Ws_1, Ws_2, G, H, C'_D, C_c, and C'_c. Such quantities as A_f, R, E_s, D, C_o, etc., may enter the chine program as parameters; that is, such quantities are constant during one run, but may be varied from one run to another.

$$\frac{dQ_1}{dt} = gr_1(E_R - P_1 - h_{L_1}) \qquad r_1 = \frac{A}{L_1} \tag{III-1}$$

$$\frac{dQ_3}{dt} = gr_3(P_7 - P_8 - h_{L_3}) \qquad r_3 = \frac{A}{L_3} \tag{III-2}$$

$$\frac{dQ_5}{dt} = gr_5(P_{14} - P_{15} - h_{L_5}) \qquad r_5 = \frac{A}{L_5} \tag{III-3}$$

$$\frac{dQ_2}{dt} = gr_2(Ws_1 - P_2 - h_{L_2}) \qquad \begin{cases} r_2 = \dfrac{A_r}{R}, & Ws_1 \geq E_s \\ r_2 = \dfrac{A_r}{Ws_1 - E_s + R}, & Ws_1 < E_s \end{cases} \tag{III-4}$$

$$\frac{dQ_4}{dt} = gr_4(Ws_2 - P_9 - h_{L_4}) \qquad \begin{cases} r_4 = \dfrac{A_r}{R}, & Ws_2 \geq E_s \\ r_4 = \dfrac{A_r}{Ws_2 - E_s + R}, & Ws_2 < E_s \end{cases} \tag{III-5}$$

$$Q_1 + Q_2 = Q_3 \tag{III-6}$$

$$Q_3 + Q_4 = Q_5, \text{ or } Q_1 + Q_2 + Q_4 = Q_5 \tag{III-7}$$

$$\frac{dQ_1}{dt} + \frac{dQ_2}{dt} = \frac{dQ_3}{dt} \quad \text{(by differentiation of III-6)} \tag{III-8}$$

$$\frac{dQ_3}{dt} + \frac{dQ_4}{dt} = \frac{dQ_5}{dt}, \text{ or } \frac{dQ_1}{dt} + \frac{dQ_2}{dt} + \frac{dQ_4}{dt} = \frac{dQ_5}{dt} \tag{III-9}$$

Substituting from (III-1), (III-4), and (III-2), Eq. (III-8) becomes

$$r_1(E_R - P_1 - h_{L_1}) + r_2(Ws_1 - P_2 - h_{L_2}) = r_3(P_7 - P_8 - h_{L_3}) \tag{III-10}$$

Substituting from (III-1), (III-4), III-5), and (III-3), Eq. (III-9) becomes

$$r_1(E_R-P_1-h_{L_1}) + r_2(Ws_1-P_2-h_{L_2}) + r_4(Ws_2-P_9-h_{L_4}) = r_5(P_{14}-P_{15}-h_{L_5}) \tag{III-11}$$

$$P_1 - P_2 = T_{1,2} \quad \text{and} \quad P_1 - P_7 = T_{1,7} \tag{III-12}$$

$$P_8 - P_9 = T_{8,9} \quad \text{and} \quad P_8 - P_{14} = T_{8,14} \tag{III-13}$$

Using (III-12), eliminate P_2 and P_7 from (III-10) to obtain

$$P_8 = \alpha P_1 + \beta, \text{ in which} \tag{III-14}$$

$$\alpha = \frac{r_1 + r_2 + r_3}{r_3}, \text{ and} \tag{III-15}$$

$$\beta = -T_{1,7} - h_{L_3} - \frac{r_1}{r_3}(E_R - h_{L_1}) - \frac{r_2}{r_3}(Ws_1 + T_{1,2} - h_{L_2}). \tag{III-16}$$

Using (III-12) and (III-13), eliminate P_2, P_9, P_{14} from (III-11) to obtain

$$P_8 = \theta P_1 + \psi, \text{ in which} \tag{III-17}$$

$$\theta = \frac{-r_1 - r_2}{r_4 + r_5}, \text{ and} \tag{III-18}$$

$$\psi = \frac{r_1}{r_4 + r_5}(E_R - h_{L_1}) + \frac{r_2}{r_4 + r_5}(Ws_1 + T_{1,2} - h_{L_2}) + \frac{r_4}{r_4 + r_5}(Ws_2 + T_{8,9} - h_{L_4}) + \frac{r_5}{r_4 + r_5}(P_{15} + h_{L_5} + T_{8,14}) \tag{III-19}$$

Solving (III-14) and (III-17) simultaneously yields

$$P_1 = \frac{\psi - \beta}{\alpha - \theta} \tag{III-20}$$

Knowing P_1 from (III-20), P_8 may be obtained from either (III-14) or 17. P_2, P_7, P_9, and P_{14} may then be evaluated by use of (III-12) and (III-13). To evaluate (III-20), it is first necessary to determine the head losses h_L and te losses T in (III-16) and (III-19).

Head Losses are determined by Eqs. (III-21) through (III-31), as follows:

$$h'_{L_1} = \left[K_1 + K'_1 (f_c L_c + f_s L_s)\right] |Q_1| Q_1 \tag{III-21}$$

$$K_1 = \frac{1 + \Sigma K}{2gA^2}, \text{ with } \Sigma K = K_{tr} + K_e + K_s + K_{ts} + K_{hb} + K_{vb} + K_{ot} \tag{III-22}$$

$$K'_1 = \frac{1}{2gDA^2}$$

$$f_c = F_c(N_R) = F_c\left(\frac{VD}{\nu}\right), \quad \frac{e_c}{D} = \text{constant} \qquad \text{(III-23)}$$
$$f_s = F_s(N_R) = F_s\left(\frac{VD}{\nu}\right), \quad \frac{e_s}{D} = \text{constant}$$

Tables of values for f_c and f_s are needed for machine runs; see, for example, Tables 1, 2, and 4 herewith. Similarly, for all other friction factors, f_3, f_4, f_5, all of which are functions of N_R, and same tables may be used.

$$h_{L_3} = K_1' f_3 L_3 |Q_3| Q_3 \qquad \text{(III-24)}$$

$$h_{L_5} = K_1' f_5 L_5 |Q_5| Q_5 \qquad \text{(III-25)}$$

$$h_{L_2} = \left(C_r + K_2 f_2 + \frac{1}{2gC_o^2 A_o^2}\right) |Q_2| Q_2 \qquad \text{(III-26)}$$

$$C_r = \frac{1}{2gA_r^2}, \text{ when } Q_2 \text{ is negative and } Ws_1 \geq E_s. \qquad \text{(III-27)}$$

$$C_r = \frac{1 + K_e'}{2gA_r^2}, \text{ when } Q_2 \text{ is positive and } Ws_1 > E_s.$$

$$C_r = 0, \text{ when } Q_2 \text{ is positive and } Ws_1 = E_s.$$

$$C_r = 0, \text{ when } Ws_1 < E_s, \text{ irrespective of sign of } Q_2.$$

$$K_2 = \frac{R}{2gD_r A_r^2}, \text{ when } Ws_1 \geq E_s. \qquad \text{(III-28)}$$

$$K_2 = \frac{Ws_1 - E_s + R}{2gD_r A_r^2}, \text{ when } Ws_1 < E_s.$$

$$h_{L_4} = \left(C_r + K_4 f_4 + \frac{1}{2gC_o^2 A_o^2}\right) |Q_4| Q_4 \qquad \text{(III-29)}$$

$$C_r = \frac{1}{2gA_r^2}, \text{ when } Q_4 \text{ is negative and } Ws_2 \geq E_s. \qquad \text{(III-30)}$$

$$C_r = \frac{1 + K_e'}{2gA_r^2}, \text{ when } Q_4 \text{ is positive and } Ws_2 > E_s.$$

$$C_r = 0, \text{ when } Q_4 \text{ is positive and } Ws_2 = E_s.$$

$$C_r = 0, \text{ when } Ws_2 < E_s, \text{ irrespective of sign of } Q_4.$$

$$K_4 = \frac{R}{2gD_r A_r^2}, \text{ when } Ws_2 \geq E_s. \qquad \text{(III-31)}$$

$$K_4 = \frac{Ws_2 - E_s + R}{2gD_r A_r^2}, \text{ when } Ws_2 < E_s.$$

Tee Losses are evaluated in accordance with the following summarization of Cases A through E (Eqs. (III-32) through (III-39)). The coefficients C_D, C_D', C_c, and C_c' are evaluated from tables; see, for example, Table 5 herewith. For ease in writing, let $h_{v1} = \frac{V_1^2}{2g}$, $h_{v3} = \frac{V_3^2}{2g}$, and $h_{v5} = \frac{V_5^2}{2g}$, and recall that $T_{1,2} = P_1 - P_2$, $T_{1,7} = P_1 - P_7$, etc., as indicated by Eqs. (III-12) and (III-13).

Case A - Tee losses ignored.

$$P_1 = P_2 = P_7 \text{ and } P_8 = P_9 = P_{14}$$

Case B - Q_1 and Q_3 are both positive and Q_2 is negative.
Q_3 and Q_5 are both positive and Q_4 is negative.

$$T_{1,2} = C_D\, h_{v_1} \qquad T_{8,9} = C_D\, h_{v_3} \qquad \text{(III-32)}$$

$$T_{1,7} = C_D'\, h_{v_1} \qquad T_{8,14} = C_D'\, h_{v_3} \qquad \text{(III-33)}$$

$$C_D \text{ and } C_D' = \text{function}\left|\frac{Q_2}{Q_1}\right| \qquad C_D \text{ and } C_D' = \text{function}\left|\frac{Q_4}{Q_3}\right| \qquad \text{Table}$$

Case C - Q_1 and Q_3 are negative and Q_2 is negative.
Q_3 and Q_5 are negative and Q_4 is negative.

$$T_{1,2} = (C_D - C_D')\, h_{v_3} \qquad T_{8,9} = (C_D - C_D')\, h_{v_5} \qquad \text{(III-34)}$$

$$T_{1,7} = -C_D'\, h_{v_3} \qquad T_{8,14} = -C_D\, h_{v_5} \qquad \text{(III-35)}$$

$$C_D \text{ and } C_D' = \text{function}\left|\frac{Q_2}{Q_3}\right| \qquad C_D \text{ and } C_D' = \text{function}\left|\frac{Q_4}{Q_5}\right| \qquad \text{Table}$$

Case D - Q_1 and Q_3 are positive and Q_2 is positive.
Q_3 and Q_5 are positive and Q_4 is positive.

$$T_{1,7} = C_c'\, h_{v_3} \qquad T_{8,14} = C_c'\, h_{v_5} \qquad \text{(III-36)}$$

$$T_{1,2} = (C_c' - C_c) h_{v_3} \qquad T_{8,9} = (C_c' - C_c)\, h_{v_5} \qquad \text{(III-37)}$$

$$C_c \text{ and } C_c' = \text{function}\left|\frac{Q_2}{Q_3}\right| \qquad C_c \text{ and } C_c' = \text{function}\left|\frac{Q_4}{Q_5}\right| \qquad \text{Table}$$

Case E - Q_1 and Q_3 are negative and Q_2 is positive.
Q_3 and Q_5 are negative and Q_4 is positive.

$$T_{1,2} = -C_c\, h_{v_1} \qquad T_{8,9} = -C_c\, h_{v_3} \qquad \text{(III-38)}$$

$$T_{1,7} = -C_c'\, h_{v_1} \qquad T_{8,14} = -C_c'\, h_{v_3} \qquad \text{(III-39)}$$

$$C_c \text{ and } C_c' = \text{function}\left|\frac{Q_2}{Q_1}\right| \qquad C_c \text{ and } C_c' = \text{function}\left|\frac{Q_4}{Q_3}\right| \qquad \text{Table}$$

As long as the form of the equations and the method of evaluation of tee losses do not change, any set of numerical values may be used for the coefficients in the machine program.

Acceleration heads are defined as follows:

$$= \frac{L_1}{gA}\frac{dQ_1}{dt}\ ; \quad h_{a_3} = \frac{L_3}{gA}\frac{dQ_3}{dt}\ ; \quad h_{a_5} = \frac{L_5}{gA}\frac{dQ_5}{dt} \qquad \text{(III-40)}$$

$$= \frac{R_2}{gA_r}\frac{dQ_2}{dt} \qquad \begin{cases} R_2 = R, \text{ when } Ws_1 \geq E_s. \\ R_2 = Ws_1 - E_s + R, \text{ when } Ws_1 < E_s. \end{cases} \qquad \text{(III-41)}$$

$$= \frac{R_4}{gA_r}\frac{dQ_4}{dt} \qquad \begin{cases} R_4 = R, \text{ when } Ws_2 \geq E_s. \\ R_4 = Ws_2 - E_s + R, \text{ when } Ws_2 < E_s. \end{cases} \qquad \text{(III-42)}$$

Water surfaces in surge tanks depend upon following equations as well as ers:

$$s_1) \over t \;=\; -\frac{Q_2}{A_{f_2}} \qquad \begin{cases} A_{f_2} = A_f, \text{ when } Ws_1 > E_s. \\ A_{f_2} = A_f, \text{ when } Ws_1 = E_s \text{ and } Q_2 \text{ is } - . \\ A_{f_2} = A_r, \text{ when } Ws_1 = E_s \text{ and } Q_2 \text{ is } + . \\ A_{f_2} = A_r, \text{ when } Ws_1 < E_s. \end{cases} \qquad \text{(III-43)}$$

$$s_2) \over t \;=\; -\frac{Q_4}{A_{f_4}} \qquad \begin{cases} A_{f_4} = A_f, \text{ when } Ws_2 > E_s. \\ A_{f_4} = A_f, \text{ when } Ws_2 = E_s \text{ and } Q_4 \text{ is } - . \\ A_{f_4} = A_r, \text{ when } Ws_2 = E_s \text{ and } Q_4 \text{ is } + . \\ A_{f_4} + A_r, \text{ when } Ws_2 < E_s. \end{cases} \qquad \text{(III-44)}$$

At Turbine end of the surge system; the following relationships must also satisfied:

$$G = F_g(t) \qquad \text{(III-45)}$$

Gives the gate opening as a function of time. Either given by manufacturer must be assumed. The function enters the machine program as a table; ce, it may have any form.

$$H = F(G, Q_5) \qquad \text{(III-46)}$$

Turbine-performance curves enter the machine program as a table; see example, Table 3 herewith. In program, G and Q_5 are the independent

variables and H is the dependent variable; that is, H is computed when G and Q_5 are known.

$$H = P_{15} + \frac{V_5^2}{2g} - Tw, \text{ in which} \quad \text{(III-47)}$$

$$P_{15} = P_{14} - h_{L_5} - h_{a_5}. \quad \text{(III-48)}$$

$$H = F_q (Q)_{Bhp} = \text{constant} \quad \text{(III-49)}$$

This function used only for stability studies; see, for example, Figs. 11 and 12 herewith. These curves enter the program as tables.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

TWO METHODS TO COMPUTE WATER SURFACE PROFILES

oe M. Lara,[1] A. M. ASCE and Kenneth B. Schroder,[2] A. M. ASCE

SYNOPSIS

vo methods derived by the authors in their work with the Bureau of .mation are presented for the computation of water surface profiles used ie development of tail water rating curves and backwater profiles. Ex- es of each method are included which show the computational procedure.

INTRODUCTION

e derivation of water surface profiles serves two major purposes in the u of Reclamation; one—to determine tail water rating curves, and two— ce the backwater curve above a dam. Tail water curves are used in the n of power-plants, pumping plants, and energy dissipators such as still- sins. In the design of large dams, these curves also furnish useful nation for making stability and stress analyses. A primary use of the vater curve above a dam is for reservoir land acquisitions and ients. Backwater information is also necessary in the design of new fa- s such as pumping plants, bridges, powerplants, or other structures d along or above the reservoir.

e methods for computing water surface profiles developed herein are red to as Method A and Method B. Method A can be used in computing es for channels in which the flow distance between sections is equal for gments of the flow. Method B differs from Method A in that it is es- ly adaptable to conditions where the flow travel paths are longer for the channel than for the overbank portion. Both methods, however, are based roper hydraulic analyses of the conveyance capacities of each cross- nal subdivision. Eddy loss corrections and velocity head changes can e taken into account by either method.

Discussion open until September 1, 1959. To extend the closing date one month, ritten request must be filed with the Executive Secretary, ASCE. Paper 1997 is t of the copyrighted Journal of the Hydraulics Division, Proceedings of the erican Society of Civil Engineers, Vol. 85, No. HY 4, April, 1959.

dr. Engr., Bureau of Reclamation, Denver, Colo.
dr. Engr., Bureau of Reclamation, Denver, Colo.

Theory

The underlying theory of each method is based on Bernoulli's Energy Equation which is repeated for convenience in the following discussion of it application to water surface profile computations.

The conditions of an open channel reach in steady, non-uniform flow are diagrammatically illustrated in Fig. 1. The energy balance equation is written:

$$Z_2 + d_2 + h_{v_2} = Z_1 + d_1 + h_{v_1} + h_f + \text{other losses}$$

where

Z_2 = streambed elevation referenced to a given datum (upstream section

d_2 = depth of flow at upstream section

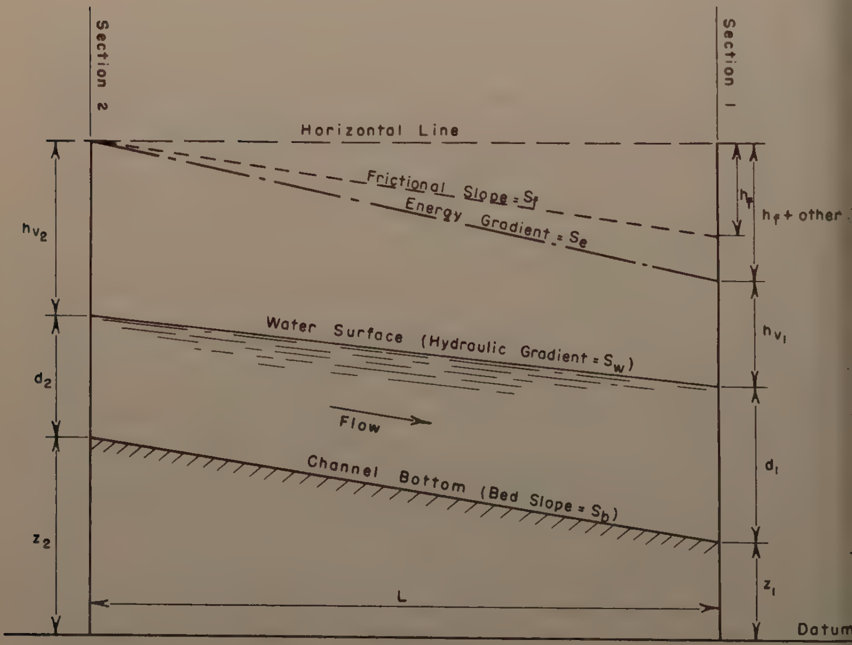
h_{v_2} = velocity head of upstream section

Z_1 = streambed elevation referenced to a given datum (downstream section)

d_1 = depth of flow at downstream section

h_{v_1} = velocity head of downstream section

h_f = friction head loss



ENERGY OF OPEN-CHANNEL FLOW

FIGURE 1

ther losses = eddy, bend, and bridge pier losses.

ne two methods use basically the following hydraulic relationships to ate the various components of the energy equation.

$$Q = AV, \; V = Q/A, \; h_v = \frac{V^2}{2g}$$

$$h_f = L \left(\frac{Qn}{1.486 \, AR^{2/3}} \right)^2$$

e

= discharge in cubic feet per second

= cross-sectional area in square feet

= cross-sectional velocity in feet per second

= gravitational acceleration in feet per second squared

= reach length between sections in feet

= roughness coefficient representing the reach

= hydraulic radius equal to cross-sectional area divided by the wetted perimeter

is noted from above that Manning's equation is used to compute h_f. rally, Manning's discharge equation is written as follows:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

e S is the slope of the energy gradient.

es

ne total losses in the energy balance equation consist of the friction head, ldy loss, bend loss, and bridge pier loss.
ne friction head can be determined by applying Manning's equation in s of the friction slope, S_f:

$$S_f = \left(\frac{Qn}{1.486 \, AR^{2/3}} \right)^2$$

otal friction head for a reach then can be resolved by the following ion:

$$h_f = L \left(\frac{S_{f_1} + S_{f_2}}{2} \right)$$

e S_{f_1} and S_{f_2} are the friction slopes computed by the preceding equation the representative hydraulic properties: n, A, and R for Sections 1 and

simplified and practical concept of eddy loss can best be understood by ing the energy losses incident to the conversion of kinetic head to static and vice versa in open channel flow. While there are, at present, no ds for measuring these losses, it is known they do occur and manifest

themselves usually in the form of turbulence. Observations of channel flow where there are abrupt changes in velocity clearly display turbulence by the presence of boils and vortexes. This is particularly evident when there is significant change in cross-sectional area.

It has been a general practice in the Bureau of Reclamation to compute th eddy loss as a 10-per cent correction of the difference in velocity heads (Δh, when the static head is converted to kinetic head and 50 per cent for a revers condition. Thus, when the value of $h_{v_1} - h_{v_2}$ is positive, a 10-per cent correction is applied, and for negative values a 50-per cent correction is used.

Departures from the 50-per cent value quoted above may be necessary, particularly, in cases where extreme changes in velocity occur in the channe reach. This condition results in transitions from a very narrow section to a wide section. Large standing waves are formed, which indicates the flow is bordering critical stage. As the flow progresses downstream, the energy is dissipated in turbulence and upon reaching the downstream section, it revert to a more tranquil state. In some of these cases, the eddy loss could reasonably be increased to as much as 100 per cent of the change in velocity heads By always applying a 50-per cent correction in the computations, a situation can result showing a higher water surface elevation at the downstream secti ($Z_1 + d_1 > Z_2 + d_2$, an adverse water surface slope) which is seldom encountered in natural open channel flow. To correct this condition, an adjustment can be made by increasing the percentage applied. Eddy losses, at best, are only estimates based on the judgment of the hydraulic engineer.

A search of the literature reveals that there have been no formulas developed yet for a thorough determination of bend losses. A number of formulas resulting from various studies are listed on following page.

A. $h_b = 0.21 \dfrac{\text{channel width}}{\text{inner radius}} h_v$ from Yarnell and Woodward(1)

B. "n" increased 0.001 for each 20° of curvature suggested by Scobey(2)

C. $h_b = 0.38\ h_v$ by C. H. Yen(3)

D. $h_b = \dfrac{0.0256\ v^2 \text{ x width of stream}}{\text{graphic radius of curvature}}$ by Boer and Urick(4)

There are no set criteria which have been established to make a practical evaluation of bend losses. Two methods are suggested for the determinatio of these losses; first, they could be computed from one of the formulas liste above; and second, "n" values could be modified to include bend loss effects Ordinarily, such losses determined by the first method are very theoretical and require analyses usually beyond the limits of practical dimensions of th collected field data. It is recommended, therefore, that "n" coefficients be assigned values to include any significant effects of bend losses.

The hydraulic conditions at bridge crossings require careful analysis to determine bridge pier losses. King's Handbook(5) recommends contracted and enlargement coefficients of 0.5 and 1.0, respectively, applied to the chan es in velocity heads at the approach, bridge, and emergent section. Nagler, Yarnell,(7,8) and Johnson(9) have developed formulas for computing the hea loss through bridges which may be applied under certain conditions.

Methods

od A

ais is a trial and error procedure which involves step-by-step compu-
as. The method has a very wide scope of application and can be used in eneral case involving steady, non-uniform flow. It is particularly appli- to the irregular channel in which the cross section consists of a main nel and separate overbank areas having individual "n" coefficients. Ve- head changes are taken into account by a weighting process and cor- ons can be included for eddy losses within the reach. Reach lengths esenting the flow path between sections, however, are assumed equal for nain channel and overbank areas.

eferring to the diagrammatic sketch in Fig. 1, the objective is to de- ine the water surface elevation for a given discharge at Section 2 when it own at Section 1.

hen the discharge exceeds main channel capacity, the flow also inundates ood plain or overbank area. If channel bed conditions are different from of the overbank areas, separate "n" values are assigned, and the cross on is subdivided for proper hydraulic analysis.

ne areas and hydraulic radii are computed separately for each subdivision abulated as shown for Section 1 in Table 1. The conveyance capacity, K_d, to 1.486/n $AR^{2/3}$, also shown in this table, is computed for the main nel and overbank using the corresponding "n" value. Conveyance curves, s. elevation, are plotted for each section; an example is shown in Fig. 2. this method, the friction slope, S_f, is determined by the following sis:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2} \qquad \text{(Manning)}$$

e

$$K_d = \frac{1.486}{n} AR^{2/3}$$

$$Q = K_d S_f^{1/2}$$

ng for S_f

$$S_f = \left(\frac{Q}{K_d}\right)^2$$

ne K_d value used in the above equation is the total conveyance capacities e main channel and overbank areas.

ne discharges occurring in each subdivision of the cross section are de- ined as follows:

$$Q = K_d S_f^{1/2}$$

$$Q_p = K_{d_p} S_f^{1/2}$$

e

= total discharge in cfs

HYDRAULIC CHARACTERISTICS

Section No. 1 Main Channel

n = 0.030 1.486/n = 49.63

Elevation	A	R	$R^{2/3}$	Sub K_d	Total K_d
5701	14.5	0.48	0.613	440	440
5702	52	1.27	1.173	3,020	3,020
5707	263	4.99	2.920	38,000	38,000
5711	449	7.21	3.732	83,000	83,000
5713	547	8.17	4.057	110,000	110,000
5713.5	570	8.55	4.181	118,000	118,678
5714	594	8.77	4.253	125,000	129,150

Overbank Area A_1

n = 0.050 1.486/n = 29.72

5713.5	57.5	0.25	0.397	678
5714.0	192	0.62	0.727	4,150

TABLE 1

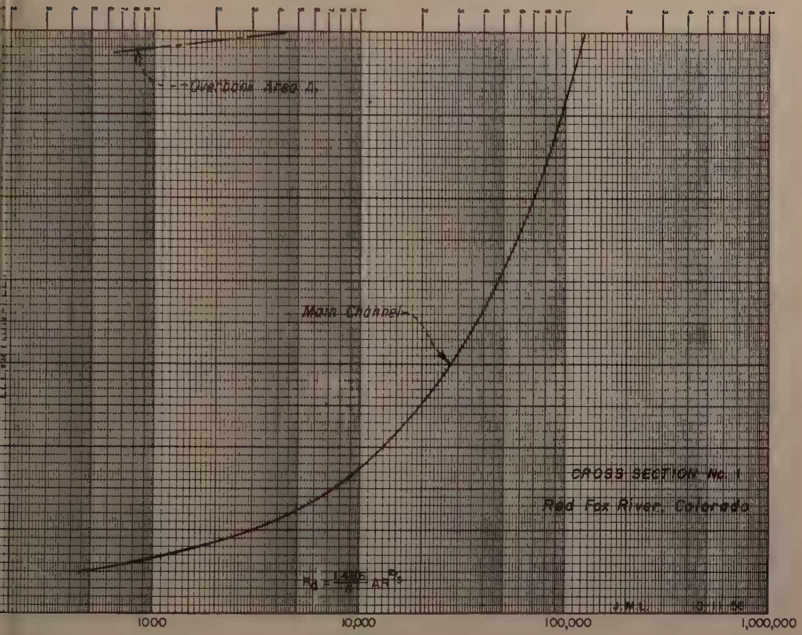


FIGURE 2

S_f = friction slope

K_d = total conveyance capacity

and K_{d_p} = corresponding elements for partial discharge and conveyance capacity of the subdivision under consideration.

ing a unit slope in the above equations, they are written thus:

$$Q = K_d \times 1 \quad \text{and} \quad Q_p = K_{d_p} \times 1$$

g one equation by the other

$$\frac{Q}{Q_p} = \frac{K_d}{K_{d_p}}$$

g for Q_p

$$Q_p = \frac{Q \times K_{d_p}}{K_d}$$

m the preceding analysis, it is noted that the K_d value, in addition to its ion as conveyance capacity, can also be designated as the discharge occ g with a slope equal to unity.

friction head, h_f, is determined by averaging the slopes at Sections 1 hen multiplying by the length as illustrated below:

$$h_f = L\left(\frac{S_{f_1} + S_{f_2}}{2}\right)$$

The velocity head, h_v, is derived by a weighting process using the partial discharges occurring in each subdivision of the cross section. This method is identical to the one used by the Corps of Engineers.(10) Velocities in each segment are computed by the basic equation, $V_p = Q_p/A_p$, where A_p is the area of the particular segment. The weighted velocity head results from the following relationship:

$$h_v = \left[\frac{\Sigma(V_p^2\, Q_p)}{2gQ}\right]$$

Eddy losses are computed from the formula $M(h_{v1} - h_{v2})$ where M is 0.50 for negative values of $h_{v1} - h_{v2}$ and 0.10 for positive values.

The following example illustrates an application of Method A using the data of a fictitious stream, the Red Fox River, Colorado, for a discharge of 11,100 cfs. The column numbers refer to the columns in Table 2.

Step 1

Enter the cross-section designation under Column 1. Column 2 shows the assumed water surface elevation. For Section 1, this has been established as 5714.0 from a predeveloped rating curve. The main channel and overbank areas and conveyance values are entered in Columns 3 and 5.

Step 2

The friction slope, S_{f_1}, is computed from the equation $S_f = (Q/K_d)^2$ as shown in Column 6. Thus, in the example, this is equal to $(11{,}100/129{,}150)^2$ 0.00755. Note that the total K_d (Column 5) is used.

Step 3

Column 9 shows the subdivisional discharges as computed by the equation

$$Q_p = \frac{Q\, K_{d_p}}{\Sigma\, K_d}$$

Substituting the values of the main channel in the example:

$$Q_p = \frac{(11{,}100)(125{,}000)}{129{,}150} = 10{,}700$$

A similar computation is made for the overbank area using its K_d value of 4,150.

Step 4

The velocities in Column 10 are obtained by dividing Column 9 by Column for each subdivision. Velocity in the main channel equals 10,700 divided by 594.

Step 5

Column 11 is computed by squaring Column 10 and multiplying by Column The values of each subdivision are totaled and entered.

SUBJECT: Silver Fox River Discharge = 50,000 cfs
COMPUTED BY: DATE CHECKED BY: DATE: SHEET 1 OF 2

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
SECTION	Reach Length L	Assumed W.S. Elevation	Area A	$K_d = \frac{1.486}{n} AR^{2/3}$	$K_d / L^{1/2}$	$\left(Q / \Sigma \frac{K_d}{L^{1/2}}\right)^2 = h_f$	$(K_d/L^{1/2})(h_f)^{1/2} = Q$	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Mean h_f	Total Loss	ΔH	Water Surface Elevation
209+40	2,260	94.67	3,390	973,000	20,500		30,000	8.85	2,350,000							94.67
	1,955		11,600	604,000	13,700		20,000	1.72	59,000							
					34,200	2.13	50,000		2,409,000	0.75						
186+80	2,260	96.90	4,140	1,070,000	22,600		28,700	6.93	1,380,000							
	1,955		13,650	739,000	16,700		21,300	1.56	52,000							
					39,300	1.62	50,000		1,432,000	0.45	+0.30	0.03	1.88	1.91	2.21	96.88
186+80	3,870	96.90	4,140	1,070,000	17,200		28,600	6.91	1,365,000							
	3,265		13,650	739,000	12,900		21,400	1.57	53,000							
					30,100	2.76	50,000		1,418,000	0.44						
148+10	3,870	100.35	4,660	1,258,000	20,200		45,700	9.80	4,390,000							
	3,265		3,520	110,200	1,920		4,300	1.22	6,400							
					22,120	5.10	50,000		4,396,400	1.37	-0.93	0.47	3.93	4.40	3.47	100.35
148+10	2,300	100.35	4,660	1,258,000	26,200		45,500	9.76	4,330,000							
	1,800		3,520	110,200	2,600		4,500	1.28	7,360							
					28,800	3.01	50,000		4,337,360	1.35						
125+10	2,300	103.40	4,520	1,350,000	28,100		21,000	4.64	453,000							
	1,800		20,700	1,642,000	38,800		29,000	1.40	56,900							
					66,900	0.56	50,000		509,900	0.16	+1.19	0.12	1.79	1.91	3.10	103.45
125+10	2,880	103.40	4,520	1,350,000	25,100		21,100	4.67	460,000							
	2,290		20,700	1,642,000	34,400		28,900	1.40	56,400							
					59,500	0.71	50,000		516,400	0.16						
96+30	2,880	104.50	4,440	1,238,000	23,100		29,800	6.71	1,340,000							
	2,290		13,100	750,000	15,700		20,200	1.54	48,000							
					38,800	1.66	50,000		1,388,000	0.43	-0.27	0.14	1.19	1.33	1.06	104.51
96+30	5,755	104.50	4,440	1,238,000	16,300		30,200	6.80	1,400,000							
	4,920		13,100	750,000	10,700		19,800	1.51	45,000							
					27,000	3.42	50,000		1,445,000	0.45						
38+75	5,755	111.10	3,840	918,000	12,100		39,300	10.22	4,020,000							
	4,920		5,900	232,000	3,310		10,700	1.81	35,000							
					15,410	10.55	50,000		4,055,000	1.26	-0.81	0.41	6.99	7.40	6.59	111.10

TABLE 3A

Step 6

The weighted velocity head, h_v, is computed from the following relationship:

$$h_v = \frac{\Sigma(v^2 Q)}{2gQ}$$

Substituting in the example:

$$\frac{3,461,730}{64.4 \times 11,100} = 4.85$$

which is entered in Column 12.

Step 7

The distance between sections is entered in Column 4.

Step 8

An elevation is assumed and entered in Column 2 for Section 2. Similar entries are made in Columns 3 and 5 as was done for Section 1 described in Step 1 above.

Step 9

The friction slope is computed as described under Step 2 and entered in Column 6. Column 7 shows the average of the consecutive slopes as follows

$$\frac{0.00735 + 0.000673}{2} = 0.00401$$

Column 8, the friction head, equals Column 7 multiplied by Column 4.

Step 10

Columns 9, 10, 11, and 12 are computed as described in Steps 3, 4, 5, an 6.

Step 11

The algebraic difference in velocity heads is entered in Column 13. In th example, this equals 4.85 - 0.43 = 4.42.

Step 12

The eddy loss Column 14 is taken as 10 per cent of Column 13 for positi values or 50 per cent for negative values. This equals (0.10)(4.42) = 0.44 in the example.

Step 13

Column 15 is the summation of Columns 8 and 14. Column 16 equals Columns 15 plus 13.

Step 14

Column 17 is obtained by adding the water surface of the downstream section (Column 17—Section 1) to the value shown in Column 16. For exam this equals 5714.0 + 6.86 = 5720.86. When the result in Column 17 is equal

nn 2 to the nearest tenth foot, the computations are begun at the next eam section.

d B

is method is also a trial and error procedure involving step compu-s. It is used in cases involving steady, nonuniform flow. However, it s from Method A in that the reach lengths representing the flow path be- sections are different for the main channel and overbank. The over-reach length could be considerably shorter. Consideration is also given variability of the hydraulic elements "n", area, and hydraulic radius as one in Method A.

ain, the sketch in Fig. 1 representing the energy equation is basic to nethod. Knowing the elevation at Section 1, the water surface elevation ction 2 is assumed and the energy equation checked.

e determination of the friction head, h_f, will be discussed first. This is nplished by applying Manning's formula as follows:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

ssing S_f as $\frac{h_f}{L}$ where L is the length between sections. Substituting

$$Q = \frac{1.486}{n} AR^{2/3} \left(\frac{h_f}{L}\right)^{1/2}$$

ituting K_d in the equation

$$Q = K_d \left(\frac{h_f}{L}\right)^{1/2}$$

ing and solving for h_f

$$h_f = \left[\frac{\frac{Q}{K_d}}{L^{1/2}}\right]^2$$

er, since there are two or more values for $K_d/L^{1/2}$ computed because differences in flow paths and conveyance capacities for the main el and overbank areas, the summation (Σ) of $K_d/L^{1/2}$ is used:

$$h_f = \left[\frac{Q}{\Sigma\left(\frac{K_d}{L^{1/2}}\right)}\right]^2$$

an friction head results from the equation $h_f = \frac{h_{f_1} + h_{f_2}}{2}$ where h_{f_1} and re the friction heads computed for Sections 1 and 2, respectively.

e discharges occurring in each sectional subdivision are computed from rmula listed below:

$$Q_p = \left(K_d/L^{1/2}\right) h_f^{1/2}$$

locity heads and eddy losses are treated the same as in Method A. application of Method B is shown below using the data of a fictitious m, Silver Fox River for a discharge of 50,000 cfs. Column numbers to the columns in Table 3.

WATER SURFACE PROFILE COMPUTATIONS – METHOD B

SUBJECT: Silver Fox River   Discharge = 50,000 cfs
COMPUTED BY: ________ DATE ________ CHECKED BY: ________ DATE: ________ SHEET 2 OF 2

1	2	3	4	5	6	7	8	9	10	11	2	13	14	15	16	17
SECTION	Reach Length L	Assumed W S Elevation	Area A	$K_d = \frac{1.486}{n} AR^{2/3}$	$K_d / L^{1/2}$	$\left(\frac{Q}{\Sigma \frac{K_d}{L^{1/2}}}\right)^2 = h_f$	$(K_d/L^{1/2})(h_f)^{1/2} = Q$	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Mean h_f	Total Loss	ΔH	Water Surface Elevation
38+75	3,875	111.10	3,840	918,000	14,750		39,200	10.20	4,080,000							
	3,295		5,900	232,000	4,040		10,800	1.83	36,000							
					18,790	7.08	50,000		4,116,000	1.28						
0+00	3,875	116.60	4,130	1,192,000	19,200		28,800	6.98	1,405,000							
	3,295		14,000	807,000	14,100		21,200	1.51	48,000							
					33,300	2.26	50,000		1,453,000	0.45	+0.83	0.08	4.67	4.75	5.58	116.63

TABLE 3B

COMPUTED BY ____________ DATE __________ CHECKED BY __________

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
STATION	Assumed Water Surface Elev.	Area A	Dist. L	$K_d = \frac{1.486}{n} AR^{2/3}$	$S_f = \left(\frac{Q}{K_d}\right)^2$	Mean S_f	Mean $S_f \times L = h_f$	Q	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Total Loss	Δ H	W.S. El.
Section 1	5714.0	594		125,000				10,700	18.0	3,460,000						
		192		4,150				400	2.08	1,730						
				129,150	0.00735			11,100		3,461,730	4.85					5714.0
			500													
Section 2	5720.9	863		230,000				5,960	6.80	276,000						
		1,170		130,000				3,370	2.88	28,000						
		1,000		25,000				650	0.65	274						
		1,500		43,000				1,120	0.75	630						
				428,000	0.000673	0.00401	2.00	11,100		304,904	0.43	+4.42	0.44	2.44	6.86	5720.86
			400													
Section 3	5721.2	1,832		360,000				11,000	6.00	396,000						
		115		2,700				70	0.61	26						
		185		1,250				30	0.16	8						
				363,950	0.00108	0.000877	0.35	11,100		396,034	0.56	-0.13	0.07	0.42	0.29	5721.15
			100													
Section 4	5721.2	1,300		158,000	0.00493	0.00301	0.30	11,100	8.54		1.13	-0.57	0.27	0.57	0.00	5721.20
			100													
Section 5	5722.1	1,681		220,000	0.00254	0.00374	0.37	11,100	6.60		0.68	+0.45	0.05	0.42	0.87	5722.07
			400													
Section 6	5722.8	1,018		174,000				10,900	10.7	1,250,000						
		130		3,380				190	1.46	405						
		20		161				10	0.50	3						
				177,541	0.00391	0.00328	1.31	11,100		1,250,408	1.75	-1.07	0.54	1.85	0.78	5722.85
			500													
Section 7	5725.3	844		124,000				10,500	12.5	1,640,000						
		98		1,400				110	1.12	138						
		220		5,800				490	2.23	2,440						
				131,200	0.00714	0.00553	2.77	11,100		1,642,578	2.30	-0.55	0.28	3.05	2.50	5725.35

TABLE 2

Step 1

Enter the pertinent data in the heading of Table 3. In the example, these are the Silver Fox River water surface profile computations for a discharg of 50,000 cfs.

Step 2

Enter the station number 209+40 under Column 1.

Step 3

Under Column 2, the main channel and overbank reach lengths between sections are entered. These are noted as 2,260 and 1,955 feet between Sections 209+40 and 186+80 for the main channel and overbank reaches.

Step 4

The known water surface elevation of 94.67 is entered in Column 3 for Section 209+40. Areas of each subdivision are entered in Column 4.

Step 5

K_d values of 973,000 and 604,000 are entered in Column 5 for the main channel and overbank area, respectively.

Step 6

Column 6 is determined by dividing the individual values of Column 5 by the square root of Column 2. For example, the main channel value is com ed as follows:

$$\frac{K_d}{L^{1/2}} = \frac{973{,}000}{(2{,}260)^{1/2}} = 20{,}500$$

Step 7

The friction head, h_f, in Column 7 is found by squaring the quantity of th total discharge divided by the summation of the values in Column 6, as follows:

$$h_f = \left(Q/\Sigma \frac{K_d}{L^{1/2}} \right)^2 = \left(\frac{50{,}000}{34{,}200}\right)^2 = 2.13$$

Step 8

The subdivisional flows listed in Column 8 are computed by multiplying the individual values of Column 6 by the square root of Column 7. For example, the overbank discharge is computed as follows: $13{,}700 \times 2.13^{1/2} =$ 20,000.

Step 9

Columns 9, 10, and 11 are determined as shown in Steps 4, 5, and 6 und Method A.

10

ne remainder of the columns are omitted since Section 209+40 is the be-
ng section. The next step, therefore, consists of assuming the water
ice elevation for the next upstream Section 186+80. This has been as-
ed as 96.90 in the example. Similar computations are made through
mn 11 as described for Section 209+40. The velocity head, h_f, for Section
30 computes to 0.45. The algebraic difference in velocity heads is
red in Column 12 as in the example $h_{v1} - h_{v2} = 0.75 - 0.45 = +0.30$.

11

ddy losses in Column 13 are determined as in Step 12 of Method A.

12

olumn 14 is the mean friction head loss, h_f, computed by averaging the
es in Column 9 as shown in the following computations:

$$h_f = \frac{2.13 + 1.62}{2} = 1.88$$

13

olumn 15 is equal to the total of Columns 13 and 14. Column 16 is the
oraic sum of Columns 12 and 15.

14

he water surface elevation in Column 17 is calculated by adding Column
the preceding water surface elevation (Section 209+40) of Column 17. In
xample, this amounts to 94.67 + 2.21 = 96.88. When a balance to the
est tenth foot is obtained between Columns 17 and 3, the computations
omplete.
he next step involves a computation of the friction head, h_f, between
ons 186+80 and 148+10 (next upstream section) using the lengths of 3,870
,265 as shown for the main channel and overbank. The water surface
tion at Section 148+10 is then assumed and a new cycle of computations
gun using the same process outlined above.

BIBLIOGRAPHY

oodward, Sherman M., "Flow of Water around 180° Bends," Technical
ulletin No. 526, October 1936, U. S. Department of Agriculture.

obey, Fred C., "The Flow of Water in Flumes," Technical Bulletin No.
93, U. S. Department of Agriculture.

en, C. H., and J. W. Howe, "Effects of Channel Shape on Losses in a
anal Bed," Civil Engineering, January 1942.

oer, B. C., and R. H. Urick, (title unknown). An experimental study of a
nd of the Iowa River made in 1913-1914. Quoted from page 627, Volume
09, Transactions, American Society of Civil Engineers, 1944.

ng, Horace W., revised by Ernest F. Brater, "Handbook of Hydraulics,"
ourth Edition, McGraw-Hill Book Company, 1954.

6. Nagler, F. A., "Obstruction of Bridge Piers to the Flow of Water," Volume 82, Transactions, American Society of Civil Engineers, 1918.

7. Yarnell, David L., "Bridge Piers as Channel Obstructions," Technical Bulletin No. 442, U. S. Department of Agriculture.

8. Yarnell, David L., "Pile Trestles as Channel Obstructions," Technical Bulletin No. 429, U. S. Department of Agriculture.

9. Johnson, Hollister, "Computation of Peak Discharge by the Contracted-Opening Method," unpublished report, U. S. Department of the Interior, Geological Survey. A similar report has been published by the U. S. Department of the Interior, "Computation of Peak Discharge at Contractions," Geological Survey Circular No. 284, 1953, C. E. Kindsvator, R. W. Carter, and H. J. Tracy.

10. "Hydrologic and Hydraulic Analyses—Computation of Backwater Curves in River Channels," Engineering Manual, Department of the Army, Corp of Engineers, Part CXIV, Chapter 9, May 1952.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

'RANSITION FROM LAMINAR TO TURBULENT FLOW IN A PIPE[a]

Closure by M. R. Carstens

. R. CARSTENS,[1] A. M. ASCE.—The discussion(1) by Prof. Robertson is rough review of the paper in which valid questions were posed concerning nterpretation of the experimental results.

he most important question of the discussion concerned the propriety of ; the Blasius boundary layer in the discussion of stability of the temporal-'veloping boundary layer in a pipe. Specifically, the question was raised whether or not boundary-layer stability studies by Tatsumi(2) would be applicable. Neither the Blasius boundary layer nor the Tatsumi bounda-yer is the time developing boundary layer in pipe, since the Blasius dary layer applies to steady-flow over a flat plate without a pressure ient and the Tatsumi boundary layer applies to steady flow in a pipe in nlet region. In spite of the differences of origin, the Tatsumi boundary r and the Blasius boundary layer are remarkably similar, as shown in I in which y is the distance from the boundary. Since the second deriva-of v with respect to y is important in stability analyses, velocity profiles appear to be quite similar as in Fig. I but result in radically different ral stability curves. Thus, further evidence of the remarkable similarity een the Tatsumi and Blasius boundary layers is shown in Fig. II in which itral stability curve for the Tatsumi boundary layer is superposed upon eutral stability curve of the Blasius boundary layer. Therefore, there ars to be no intrinsic merit in using the Tatsumi boundary layer for tative discussion of a case for which the stability analysis is not availa-On the other hand, the advantage of using the Blasius boundary layer for parison is that the stability analyses are much more comprehensive than ny other boundary-layer velocity profile.

he comparison of the stability of the temporally developing boundary layer pipe can also be made with the stability of the Polhausen boundary .(3) In Fig. III the plotted points were obtaineu from an analog computer ion of the Navier-Stokes equation with boundary conditions and physical tants of Run 29. The analog computer solution for the velocity profiles n in Fig. III is in the range of boundary-layer thickness for which tran-1 to turbulence was observed. The ratio of the displacement thickness, the momentum thickness, Θ, is about 2.43 for the four velocity profiles n in Fig. III. This value of δ^*/Θ corresponds to a Polhausen shape meter of 3. Stability analysis of this Polhausen boundary layer indicates he critical or minimum value of the boundary-layer Reynolds number,

---

roc. Paper 1450, December, 1957, by M. R. Carstens.

rof. of Civ. Eng., Georgia Inst. of Technology, Atlanta, Ga.

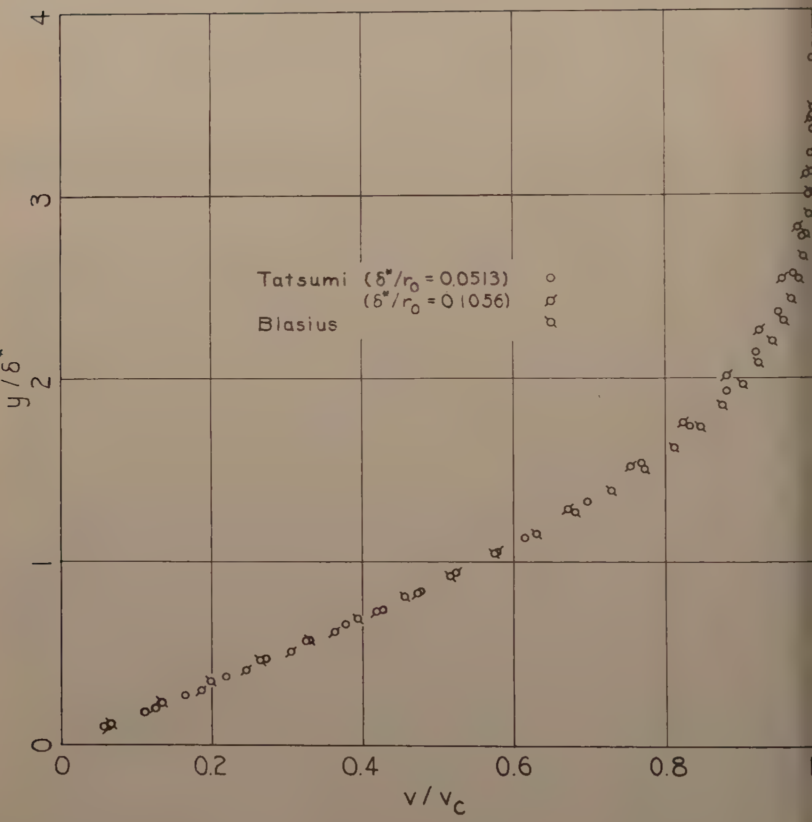
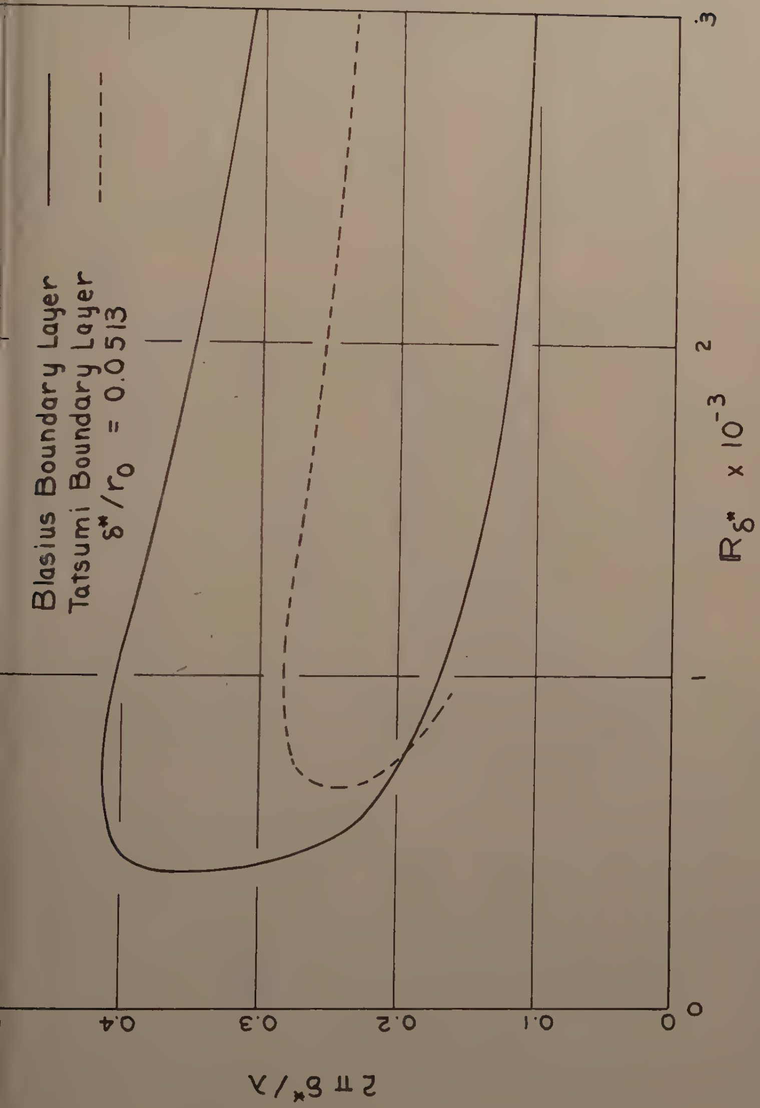


Figure I. Comparison of the Blasius and Tatsumi Boundary-Layer Velocity Profiles



gure II. Comparison of Neutral Stability Curves of the Blasius and Tatsumi Boundary Layers

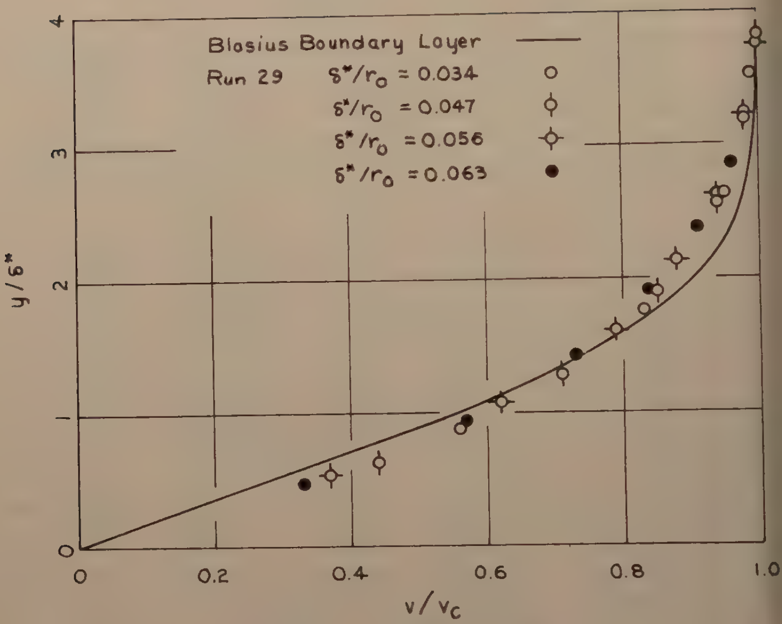


Figure III. Boundary-Layer Velocity Profiles of Run 29

$\mathbb{R}_{\delta*}$ is 3500.(4) However, in all of the experimental runs, the inception o turbulence was always observed (see Table III of the paper 1450) at a valu of $\mathbb{R}_{\delta*}$ less than 3500. This observation tends to substantiate the discuss statement "there is some evidence that a timewise developing flow may be less stable than a steady flow."

In spite of the hazards of using a borrowed solution, that is, the stabilit of the Blasius boundary layer, the general nature of the boundary layer ins bility in the time developing boundary layer of the experiments is undoubt identical to that of the borrowed solution. The fact that the borrowed stab analysis offered a rational explanation of the observed transition to turbul is indicative to the author that the neutral curve of stability of the time de veloping boundary layer must be very similar, both quantitatively and qua tatively, to the neutral stability curve for the Blasius boundary layer.

The discusser has correctly deduced the manner of obtaining the "poss neutral stability curves" which are shown in Fig. 10, that is, the disturba wave length was chosen which would pass the "possible neutral stability curve" through the experimental point having the highest transition Reyno number. The hypothesis that the disturbance wave length, λ, is proportio to the roughness height was based upon an analogy with the Karman vorte trail behind a circular cylinder. In a stable Karman vortex trail, the spa between successive vortices is proportional to cylinder diameter and inde pendent of the flow variables. The analogy is suggestive that the disturba wave length, λ, is proportional only to the roughness height or, in this ca the step height of the mismatched pipe boundaries at the junction. The m matching of the pipe ends could not be expected to be axisymmetric. Thu

disturbance wave lengths, ranging downward in magnitude from the
num wave length at the point of greatest mismatching, could be expected
ginate at the pipe junction. Only the possible neutral stability curve for
ngest disturbance wave length was plotted on Fig. 10. The other shorter
bance wave lengths originating at the pipe junction would have neutral
ity curves lying below the plotted curve. The curve defining the limits
tral stability of all disturbances would be bounded by a vertical line on
ft in Fig. 10 at a value of $R_{\delta *}$ of 420 with an upper boundary as shown in
10. The above explanation was the basis for the statement "With this
ot there is no conflict in that some of the points of turbulence inception
ated by the circular symbols on Fig. 10) are below the lower branch of
rve shown."

e question was raised in the discussion concerning the amplification of
urbance which is required to produce turbulence inception. The stability
ses of small disturbances are merely indications as to whether the dis-
ice will amplify or decay. The stability analyses do not indicate whether
nplification of a disturbance is sufficient to result in transition to turbu-
The presumption is that, when a disturbance reaches a certain magni-
transition occurs. The order of magnitude of amplification required for
tion is not a constant, as implied by the discusser, but must certainly
d upon the initial magnitude of the disturbance. For example, an infini-
al disturbance would require infinite amplification and a finite dis-
ice, which is equal or greater than the limiting magnitude, would cause
tion without amplification. Since the transition to turbulence was always
ved some distance downstream from the pipe junction, the conclusion is
ie disturbances were small (but certainly finite) disturbances that re-
l some amplification before transition occurred.

e discussion concerning Fig. 7 appears to result from a misunderstand-
A brief review of the total transition occurrence will clarify the meaning
s figure. Prior to opening of the downstream valve, the fluid is at rest
hout the pipe. At the instant of valve opening a piezometric-pressure
ent is applied to the fluid in the pipe. The unbalanced force system re-
in acceleration. The developing flow pattern can be visualized as a
l core with uniform velocity distribution and a boundary layer in which
locity varies from zero at the boundary to the core velocity. The core
ty increases linearly with time during the initial stages of the motion.
oundary layer is of zero thickness at t = 0 and increases with time.
ow is laminar throughout. Meanwhile, small disturbances are being fed
ie boundary layer downstream from the pipe junctions. There is only a
d range of core velocity, v_c, and boundary-layer thickness, $\delta*$, during
these disturbances will be amplified as shown Figs. 9 and 10. During
eriod of amplification, a disturbance from the pipe junction is amplified
turbulent burst is formed. Once a turbulence burst exists, large ad-
ents in the laminar velocity profile are effected. Since the pipe flow is
d in extent, these adjustments within the boundary layer also result in
ciable velocity changes in the central core adjacent to the turbulent
Since the concept of a laminar central core riding over a turbulent
ry layer appears to be untenable, the turbulence must penetrate into
ntral core in some manner. Whether the velocity adjustment leads to
lity in the central core or whether the disturbance of the turbulent
is sufficient to cause instability and a radial spread of turbulence is not
to the author. In any event the schematic drawing in Fig. 7, which is

based upon a study of the film strip data and pressure data as the downstrea face of the turbulent spot passed through the pipe outlet, appears to be a rational picture of the downstream face of a turbulent spot which moves toward the pipe outlet. Essentially, the same type of downstream face of a turbulent spot is reported by Schubauer and Klebanoff(5) for flow over a flat plate as follows:

> "The turbulence is, in fact, transported downstream with the free-stream velocity and the lag at the surface is due to the time required for propagation inward through the slower moving air of the laminar layer. Probably the chief significance of the slower progress near the surface is that it gives rise to an overhanging leading edge."

Two errors in the paper were pointed out in the discussion. First, the number of wave lengths between the point of disturbance origin and the point of turbulence inception would be about 4 to 37 for turbulent spot 2 and about 13 to 31 for turbulent spot 3 based on wave lengths of 2.1 r_o and 1.5 r_o, respectively. Second, the X/D value for spot 1 of Run 20 should be 12 rather than 112 in Table III. A question mark in parenthesis should be placed in Table III adjacent to X/D value of spot 1 of Run 21.

The author thanks Professor Robertson for his carefully prepared discussion of the paper and thanks Messrs. Johnson and Meeks of the Georgia Tech Analog Computer Section for their help in obtaining the solutions show in Fig. III.

REFERENCES

1. Proc. Paper 1856, November, 1958, by J. M. Robertson.
2. Tatsumi, Tomomasa, "Stability of the Laminar Inlet-Flow prior to the F mation of Poiseuille Régime," Journal of the Physical Society of Japan, Vol. 7, No. 5, Sept.-Oct., 1952, pp. 489-502.
3. Schlichting, H., Boundary Layer Theory, Pergamon Press, 1955, pp. 206 213.
4. Ibid., p. 341.
5. Schubauer, G. B. and Klebanoff, P. S., "Contributions on the Mechanics o Boundary-Layer Transition," NACA, TN 3489, September 1955.

SOME EXPERIMENTS WITH EMERGENCY SIPHON SPILLWAYS[a]

Discussion by Fred W. Blaisdell and Harold W. Humphreys

RED W. BLAISDELL,[1] M. ASCE and HAROLD W. HUMPHREYS,[2] A. M. E.—India, in addition to the countries listed by the author, has used large on spillways for the close control of reservoir levels. But only India, to vriters' knowledge, has used both "saddle" and "volute" siphons,(1) as call them. This discussion will be confined largely to the volute siphon various modifications of it.

he volute siphon might be defined as a morning glory spillway which has entrance covered by a dome. The dome is of greater diameter than the ance and extends below the crest elevation in the same manner as does op or hood of the saddle siphon. Also, like the saddle siphon, the volute on uses siphon breakers which run to the top of the dome. In contrast to saddle siphon as detailed by Mr. McBirney, the volute siphon does not e the lower leg upstream or necessarily use a device to throw the nappe e crown, nor is it necessary to submerge the exit to insure priming.

dian literature credits Shri V. Ganesh Iyar with the discovery of the e siphon in 1933 and its experimental development at Mandya in 1935-36 a siphon 2.5 feet in diameter and 11 feet high.(1,2) Two volute siphons feet in diameter with an operating head of 43 feet were constructed at conahally in 1938 as an experimental measure. Eleven siphons 18 feet in eter with a drop from the design head-water level to the invert at the way exit of 68 feet were built in 1947 at Hirebhasgar.

is not necessary to use the dome to insure that the spillway will run full r suction. Examples of this may be found in the Heart Butte, North ta, and Shade Hill, South Dakota, spillways of the U. S. Bureau of Reclaon.(3) If a siphon is defined as a spillway which requires that the liquid fted above the headpool level, then the Dakota spillways do not qualify as ns. They do qualify as siphons if the definition is:

> closed conduit, a portion of which lies above the hydraulic grade line. is results in a pressure less than atmospheric in that portion, and nce requires that a vacuum be created to start flow. A siphon utiles atmospheric pressure to effect or increase the flow of water rough it.

---

roc. Paper 1807, October, 1958, by Warren B. McBirney.

roject Supervisor, Agri. Research Service, U. S. Dept. of Agri., St. nthony Falls Hydr. Lab., Minneapolis, Minn.

ead, Hydr. Research Section, State Water Survey, Urbana, Ill.

The Dakota spillways qualify as siphons because, under full flow, the vertica legs are under pressures less than atmospheric and lie above the hydraulic grade line.

The possibility that the pressures within these spillways may be so low a to cause cavitation should be recognized. The Hirebhasgar volute siphons[1] in India are examples. These siphons consist of a dome-covered morning glory entrance 30 feet 8 inches in diameter which converges to a drop inlet shaft 18 feet in diameter, a 90 degree elbow, and a horizontal leg 18 feet in diameter. The drop in level between the crest and the exit center line is ab 57 feet—considerably in excess of absolute zero pressure head even conside ing hydraulic losses. Cavitation damage should have been expected and did occur.[1] Cavitation damage also has occurred in the Marconahally siphons. This should have been anticipated since one-sixteenth size model tests indicate prototype pressures below absolute zero over much of the drop inlet an inside of the elbow.[5]

In the Dakota spillways of the U. S. Bureau of Reclamation, the possibilit of extremely low pressures was recognized and means were taken to preven their occurrence. In the case of the Heart Butte spillway, a lip is used to cause the stream to break away from the wall of the drop inlet. This insure atmospheric pressure at that point and that negative pressures in the drop inlet will not exceed a predetermined magnitude. The drop through the Shad Hill spillway does not appear to be sufficient to cause cavitation problems.

Full flow can be achieved for spillways of the morning glory type without special priming devices if the inlet is properly formed. The writers are of the opinion that the Hungry Horse spillway[3] would flow full if the head ove the crest is raised high enough. If this were to occur, serious cavitation would be a likelihood, in view of the 487 foot drop through the spillway, unle means were taken to prevent it. Presumably the maximum head over the crest is sufficiently low and the barrel sufficiently large to insure part full flow at all times. It is important that designers recognize that morning glo spillways can flow full, in order to know what to design against if they want part full flow, or to know what to design for if they want full flow.

The Dakota spillways required no priming or depriming aids other than shaping the inlet properly. The head-discharge curves for the Heart Butte spillway presented by Peterka[6] show that the flow through the spillway is controlled by a weir at low heads. At high heads, the control is by pipe flov when the inlet is submerged and the drop inlet is full. At the higher weir heads, the falling water must partially seal off the drop inlet and the increa head must create a demand for more water than can be supplied over the crest weir at that headpool level. The excess demand is supplied by air en trainment and air flow through the spillway. This is, the writers believe, t partialization for which Mr. McBirney is looking. Partialization is achieve here without special devices.

Only relatively large spillways have been discussed up to this point. La closed conduit spillways designed to flow full, like the Heart Butte and Shad Hill structures, are few in number. However, the U. S. Soil Conservation Service (SCS) began building this type of spillway in relatively smaller size in the 1930's. Closed conduit self-priming spillways built annually under Public Law No. 46, 74th Congress, entitled "An Act to Provide for the Protection of Land Resources Against Soil Erosion, and for other Purposes;" a Public Law 566, 83rd Congress, known as the Watershed Protection and Fl

rention Act, number in the thousands[7] and many of them operate at de- capacity each year. The hydraulic design of these SCS spillways is tical to the design methods which should be used for the relatively larger ways of the morning glory type. The senior writer has outlined the theory iously[8] and recently has authored a series of papers[9] describing both ratory and field tests on a number of different inlet configurations. It be of interest to mention in passing that the hydraulics involved is that nt in elementary courses but that the application requires a thorough vledge of the conditions under which each of the elementary hydraulic laws applied.

great many of the SCS-designed closed conduit spillways consist of a ical drop inlet from the base of which a barrel carries the flow through dam. The slope of the barrel may be either steeper or flatter than that of ydraulic grade line. If the drop inlet is properly proportioned, the spill- will flow full whatever the barrel slope may be even if the outlet is not nerged. Pressures within the barrel will be less than atmospheric and barrel will lie above the hydraulic grade line if the barrel slope is steeper the hydraulic grade line. The spillway then acts as a siphon if the broad ition presented earlier is accepted.

series of tests on drop inlet closed conduit spillways that are pertinent is discussion was initiated in 1957 at the Urbana Hydraulic Laboratory of llinois State Water Survey under the direction of the junior writer. This way consists of a circular drop inlet five barrel diameters (5D) high by barrel diameters (5D/3) in diameter. The barrel diameter D is three es. The 30 per cent slope on which the barrel has been placed initially is iderably steeper than the hydraulic grade line slope. Water enters the inlet from the entire periphery of the crest. The head-discharge curve his spillway has the form shown in Fig. A. Section I of the head-discharge e represents weir control with the weir at the crest of the drop inlet. ion II is also weir control but air is carried through the spillway with the r. Section III represents pipe control. The head-discharge curve and the have been fully described elsewhere.[10]

he primary purpose of the Illinois State Water Survey study is to de- ine the performance of a flat plate anti-vortex device supported above crest and to develop design rules for the form and dimensions of the plate. initial flat plate is two drop inlet (riser) diameters ($2\,D_R$) in diameter three guide vanes, 120° apart, attached to the plate. The guide vanes ex- radially outward from the outside edge of the drop inlet. The solid curve ig. A is for the case where the anti-vortex plate is located $0.556D_R$ or er above the inlet crest, that is, within the range of pipe control repre- ed by Section III. When the plate was lowered into the weir control ranges I, it was found that the plate sealed off the inlet as soon as the water reached the bottom edge of the plate. This prevented access of air to lrop inlet and the flow caused the creation of a partial vacuum. The um sucked air under the plate in sufficient quantity to satisfy the demand. easing the water flow caused little or no change in headpool level until spillway flowed full of water alone. Air was sucked into the spillway in unts to maintain a vacuum just sufficient to permit the water flow to take e at constant headpool levels. The dash lines in Fig. A show the head- harge curves for successively lower elevations of the anti-vortex plate. headpool level in essentially steady, exhibiting none of the fluctuations

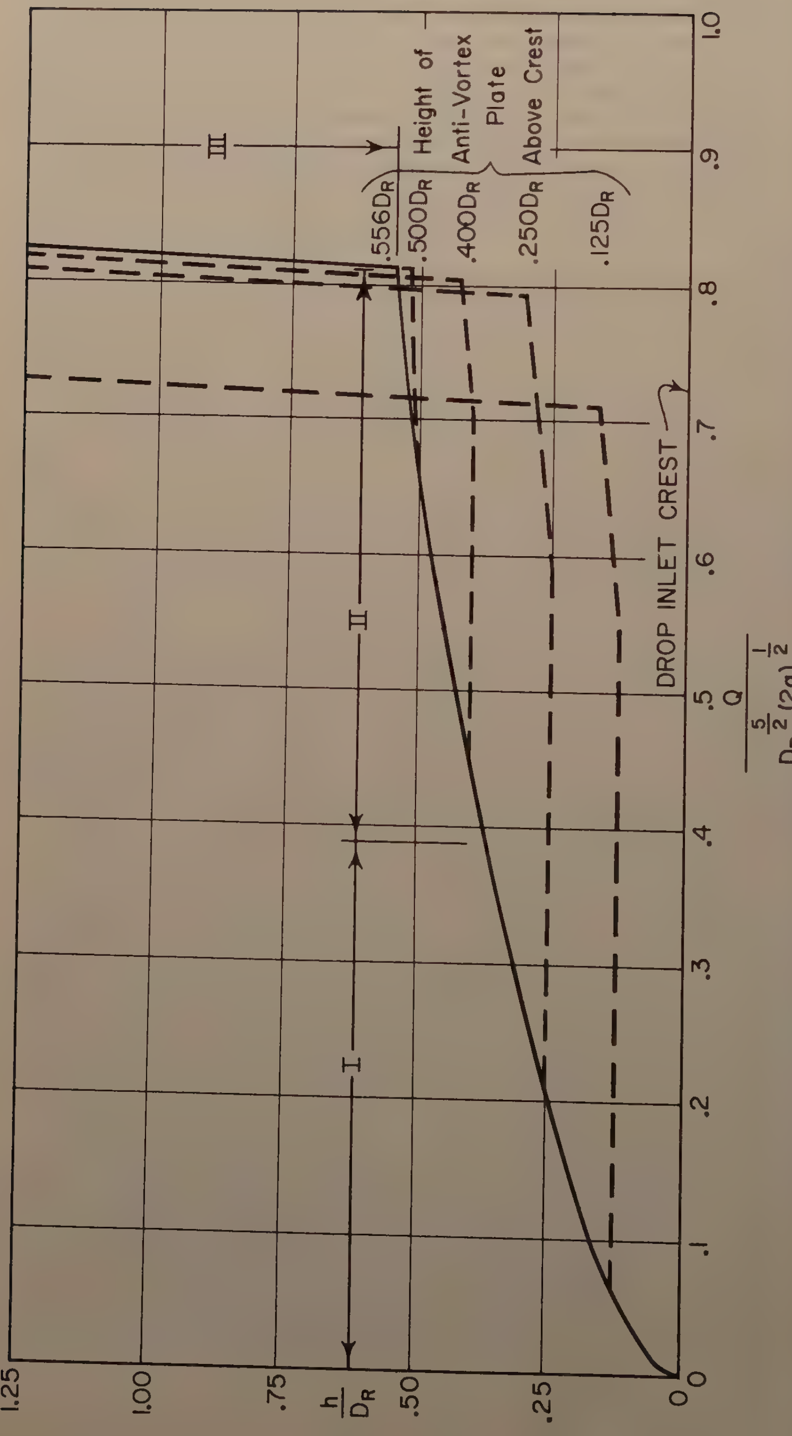


Fig. A - EFFECT OF $2D_R$ CIRCULAR PLATE POSITION ON CLOSED CONDUIT SPILLWAY HEAD-DISCHARGE RELATIONSHIP

rn by Mr. McBirney's Fig. 4. Is this not the partialization and steady pool level that Mr. McBirney is looking for? The possibility of positively electing the priming head within certain presently undefined limits by ng the flat plate at the correct height above the crest should not be overed.

o give actual figures for comparison, Mr. McBirney's spillway apparently a throat area of 1 x 0.5 = 0.5 square feet. The Illinois spillway has a el area of $(0.25)^2 \pi/4 = 0.0491$ square feet. The linear multiplication or to secure comparable priming heads is then $\sqrt{0.5/0.0491} = 3.19$. Plate nts as low as 5/8 inch caused the Illinois spillway to prime. Increasing to the scale of Mr. McBirney's tests, the priming head becomes 0.17 feet. corresponds to Mr. McBirney's full flow priming head of 0.52 feet for standard siphon design and 0.42 feet for the proposed design.

roper positioning of the flat plate anti-vortex plate used as a siphon er results in a single-valued head-discharge curve. The curves of two s shown by Mr. McBirney in Figs. 6 and 11 for the standard and proposed gns do not exist for the flat plate priming device in the positions shown in A. However, two loops can be obtained for the drop inlet entrance by exing skirts from the periphery of the cover plate down below the crest of weir. The spillway would then prime on rising stages at the same head as e skirts had not been used, but the prime would be held until the pool level hed the bottom of the skirts, or to the siphonbreaker if one is used. The ted arrangement is, of course, what is called a vortex siphon spillway in a.

he priming time, as used by Mr. McBirney for the saddle siphons, does exist for the drop inlet closed conduit spillway. Priming for the latter way takes place gradually as the headpool level rises and does not occur tively suddenly as for the saddle siphon.

he gradual increase and decrease of flow obtained with the drop inlet on also reduces the erosion damage downstream from the outlet which McBirney says results at many locations from the prime-break-prime e of intermittent operation.

he writers do not say that the drop inlet closed conduit spillway, either out or with the flat plate anti-vortex device and primer, is a complete tion to all siphon problems. They do maintain, however, that the potenties of the drop inlet siphon should not be overlooked.

REFERENCES

oglekar, D. V., Guha, S. K., and Luthra, S. D. L., "High Head Siphons, heir Behavior and Characteristics." Proceedings of the Sixth General leeting, International Association for Hydraulic Research, The Hague, ol. 3, pp. C2-1-C2-10, 1955.

ao, N. S. Govinda, "Design of Siphons," Irrigation and Power Journal ndia) Vol. XI, No. 4, October 1954.

radley, Joseph N., "Prototype Behavior," Transactions, ASCE, Vol. 121, 956, pp. 312-333.

lossary: Sewage and Water Control Engineering, American Society of ivil Engineers, New York, 1949.

5. Rao, N. S. Govinda, "Design of Siphons," Irrigation and Power Journal (India) Vol. XII, No. 4, October 1955.

6. Peterka, Alvin J., "Performance Tests on Prototype and Model," Transactions, ASCE, Vol. 121, 1956, p. 385, Fig. 8.

7. U. S. Dept. of Agriculture, "Agricultural Statistics 1956," Washington, D. C.

8. Blaisdell, Fred W., "Hydraulic Fundamentals of Closed Conduit Spillways," Proceedings-Separate No. 354, ASCE, November 1953.

9. Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Parts II through VII, Results of Tests on Several Forms of the Spillway," St. Anthony Falls Hydraulic Laboratory Technical Paper No. 18, Series B, Minneapolis, Minn., March 1958;

Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Part VIII, Miscellaneous Laboratory Tests," St. Anthony Falls Hydraulic Laborato Technical Paper No. 19, Series B, Minneapolis, Minn., March 1958; and

Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Part X, Th Hood Inlet," St. Anthony Falls Hydraulic Laboratory Technical Paper No. 20, Series B, Minneapolis, Minn., April 1958.

10. Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Part I, Theory and Its Application," St. Anthony Falls Hydraulic Laboratory Technical Paper No. 12, Series B, Minneapolis, Minn., January 1952, Revised February 1958; and

Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Parts II through VII, Results of Tests on Several Forms of the Spillway," St. Anthony Falls Hydraulic Laboratory Technical Paper No. 18, Series B, Minneapolis, Minn., March 1958.

SYNTHETIC FLOOD FREQUENCY[a]

Discussion by Max A. Kohler

MAX A. KOHLER,[1] M. ASCE.—Mr. Snyder's approach to generalized, or thetic, flood frequencies can best be described as an elaboration and im-vement of the so-called "rational method." It involves the assumption that n-year flood can be derived from the n-year rainfall even though the two nts do not necessarily bear the relation of cause and effect.

Specifically, the proposed method involves (1) determination of the n-year nfall for the basin with specified area and concentration time T_c, (2) con-sion of n-year rainfall to n-year runoff volume and (3) determination of flood peak corresponding to the runoff volume. This discussion concerns marily steps (1) and (2) of the procedure since it is believed that Snyder's any of several other available techniques are adequate for converting run-volume to corresponding peak flow.

Before taking up specific aspects of Snyder's procedure, it is perhaps best consider the basic underlying assumption that a simple relation exists be-en the n-year flood peak and the n-year rainfall. From a probability point iew, this implies that the composite probability for all factors which af-t the relation between rainfall amount and peak flow is unity. Therefore, only direct means of relation n-year rainfall and flood peaks is through istical analysis of n-year data. This means that flood-frequency data st be used in deriving the generalized relations. There is no indication on part of the author that flood-frequency data are necessary or were used. tainly there was some basis for deciding that the required relation should e "slightly higher runoff than the average" (indicated by average monthly nfall and runoff). Wouldn't one expect a different rainfall-runoff relation small basins (T_c < 4 hrs) where the major floods result from summer nderstorms than for the larger basins where such floods result from gener-ains on wet soil? It is believed the author has tended to account for this iation by an unrealistic reduction of point rainfall for T_c and drainage a. Thus he chose to derive an area-reduction relation from storm data in ference to using one properly derived from frequency data.(1) It is not r why his results indicate a flatter curve than one based on frequency , while just the opposite would be expected.

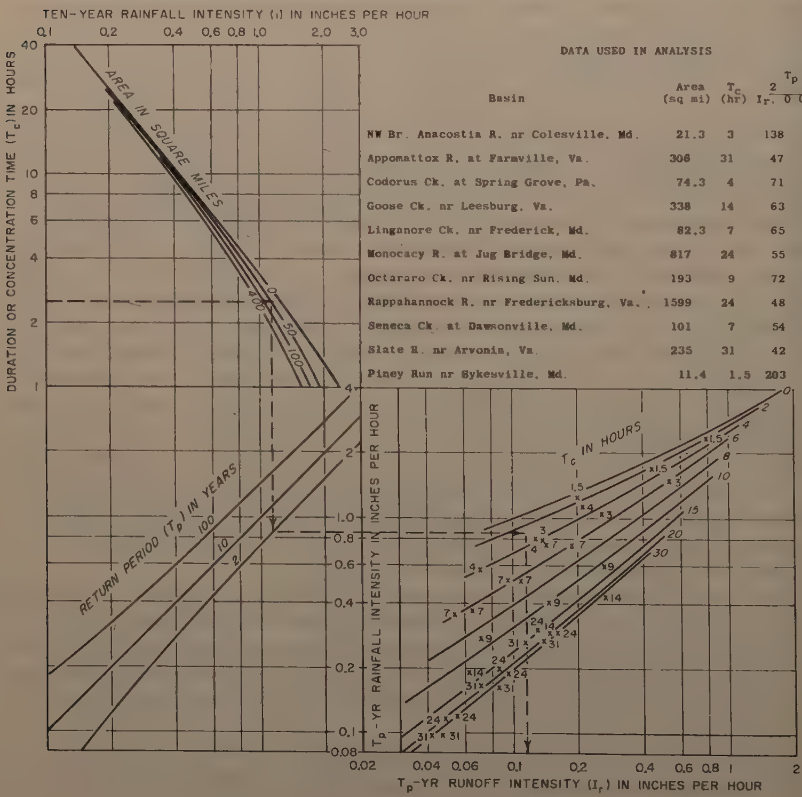
f it is visualized that Snyder's procedure constitutes a method of generaliz- or regionalizing, on the basis of existing flood-frequency data, certain lifications would appear to be appropriate. Although not absolutely neces-, it is suggested that the empirical tie between n-year rainfall and flood- be confined to the rainfall-runoff relation. In this case, the n-year

Proc. Paper 1808, October, 1958, by Franklin F. Snyder.
Chf. Research Hydrologist, U. S. Weather Bureau, Washington, D. C.

runoff can be computed from Snyder's relations for all basins with established flood-frequency curves. These values can then be related to corresponding values of n-year basin rainfall derived from the point-rainfall frequency relation (author's Fig. 2) and one of areal rainfall vs duration (T_c) and point rainfall.(1) This is the manner in which the accompanying chart was developed.

The values of peak discharge (Q_p) for various basins and return periods were obtained by Gumbel analysis with appropriate adjustment to give partial-duration data. The average rate of direct runoff (I_r) for time T_c was obtained by use of the author's formula, $I_r = Q_p/500A$, where A is the basin area in square miles.

The first part of the accompanying relation, namely, the area curves in the upper left, was derived from the 10-year rainfall curve of the author's Fig. 2. The 10-year rainfall amounts for various concentration times, or durations, were first plotted and the zero-area curve was drawn through the points. The curves for other sizes of area were then constructed on the basis of the depth-area relation(1) given in U. S. Weather Bureau Technical Paper 29. Although the depth-area relation is based on rather limited data, it is considered to be the best available for the purpose. The second part of the writer's relation, i.e., the return-period curves in the lower left, was obtained by entering the



DATA USED IN ANALYSIS

Basin	Area (sq mi)	T_c (hr)	T_p (yr) 2	10	100
			I_r, 0 001 in /hr		
NW Br. Anacostia R. nr Colesville, Md.	21.3	3	138	261	[illegible]
Appomattox R. at Farmville, Va.	306	31	47	84	[illegible]
Codorus Ck. at Spring Grove, Pa.	74.3	4	71	129	[illegible]
Goose Ck. nr Leesburg, Va.	338	14	63	150	[illegible]
Linganore Ck. nr Frederick, Md.	82.3	7	65	96	[illegible]
Monocacy R. at Jug Bridge, Md.	817	24	55	86	[illegible]
Octararo Ck. nr Rising Sun, Md.	193	9	72	151	[illegible]
Rappahannock R. nr Fredericksburg, Va.	1599	24	48	94	[illegible]
Seneca Ck. at Dawsonville, Md.	101	7	54	109	[illegible]
Slate R. nr Arvonia, Va.	235	31	42	72	[illegible]
Piney Run nr Sykesville, Md.	11.4	1.5	203	443	[illegible]

quadrant with T_c, going horizontally to the zero-area curve, then down-to the appropriate rainfall intensity (indicated by the author's Fig. 2). point so determined was labelled with the corresponding return period smooth family of curves were fitted to the points.(2)

should be pointed out that the return-period curves cannot be drawn to y fit areas of all sizes, but the errors are insignificant with respect to rrors of the procedure as a whole. The final curve family (T_c curves) erived by plotting n-year basin rainfall intensity computed from the first uadrants against corresponding values of n-year runoff intensity I_r and ng the points with T_c.

e accompanying table presents the basic data for eleven natural basins general vicinity of Washington, D. C., which were used in deriving the urves. These basins were selected because flood-frequency curves based g records of observed and synthesized(3) annual peaks were available re unit hydrographs for estimating T_c.

e application of the writer's relation for estimating n-year runoff in-y is straightforward and needs no further amplification here. The pro-es described by Snyder for obtaining T_c and for converting I_r to the n-peak flow are equally applicable when using the writer's accompanying on.

conclusion, the writer wishes to commend the author for presenting new timulating ideas on an important phase of hydrologic design.

REFERENCES

Rainfall Intensity-Frequency Regime, Part I: The Ohio Valley," U. S. eather Bureau Technical Paper No. 29, June 1957.

Iydrology for Engineers", R. K. Linsley, M. A. Kohler and J. L. H. ulhus, pp. 311-321, McGraw-Hill, 1958.

Flood Frequencies Derived from Rainfall Data," J. L. H. Paulhus and F. Miller, Proc. Paper 1451, Jour. Hydraulics Div., ASCE, Vol. 83, ecember 1957.

DIVINING RODS VERSUS HYDROLOGIC DATA AND RESEARCH[a]

Discussions by Erhard E. Dittbrenner and V. M. Yevdjevich

RHARD E. DITTBRENNER,[1] M. ASCE.—Mr. Langbein presents a scholar-
d comprehensive review of the status of the most critical need in hy-
gy. It would be difficult to add anything useful to his presentation on hy-
gy. However, when he includes "waterwitching" in hydrology—even if
as a strawman to be knocked down and used as a reference—it is time to
rate fact from fancy.
r. Langbein states: "Modern hydrology offers no support of water
ery." In a certain limited sense this statement may be accurate enough.
urther question: ". . . can we be scientific about witchery," is obviously
rical, as his implication is that since witchery is perhaps in the realm
ery, it cannot be scientific.
e answer to his question is, however, an unqualified "Yes." We can be
tific about "witchery." To do so, however, we must go to that portion of
cal science which has to do with electrical phenomena. This might seem
e it out of that portion of science which we have artifically labeled "hy-
gy." It is not important to argue the point, but it seems pertinent to
out that as we push back the frontiers of our knowledge in our artificially
ribed categories of science it becomes clear that Nature has no such
ary divisions. More and more evidence substantiates a belief perhaps
ancient than water witching—that there is a oneness or unity in the
rse.
e writer happens to be a "water witch" or dowzer. Many years ago he
nvited to try his hand by a young dowzer who suspected skepticism of the
my attitude. The dowzer pointed out that many people have the dowzer's
ilities without realizing it. One trial demonstrated the truth of that
nent and the basic reason why water witchery has persisted over the
and through the ages. Once a person experiences the phenomena which
the switch down, he will never be bothered by the scoffing of the "scien-
mind. In the writer's case, the switch broke in his hands on his first
It has never done so since, but since he never worked at the art very
this may not be very significant.
ce there had to be some scientific explanation for the phenomena, search
rature and query among those more qualified in science than himself
brought some answers. The "dry" holes can be explained by the
edge that the forces involved in the phenomena are obviously too small
ak the twig, but act on certain muscles which are thereby impelled to
about the action of the switch without volition on the part of the dowzer.

oc. Paper 1809, October, 1958, by W. B. Langbein.
dr. Engr., U. S. Bur. of Public Rds., Albany, N. Y.

The forces involved vary constantly—during the day, with the weather, and from season to season. Some experienced dowzers are aware of this. Other obviously are not. Since the dowzers are not aware of the source of the phenomena or their powers, they are unable to explain these variations.

Two scientists in England have studied this phenomena extensively. Maby and Franklin, in "The Physics of the Divining Rod," London, 1939, cover the subject comprehensively, but by no means exhaustively. The phenomena has been measured. Instruments have been devised to determine quantitatively the field in which the dowzer works to make his determinations. People have been checked to determine their powers of "divination." About one person in ten has these powers in some degree. Their investigation did not extend to the point of determining the reasons why some do and others do not have them. So far as this writer knows, no one has made any effort in this direction—in fact, since Maby and Franklin's publication, no further scientific literature of that nature on the subject has appeared to the writer's knowledg

In the writer's own experiences, a routine laboratory experiment during college required the measurement of each student's bodily electrical resistance. His resistance was way out of line with that of any other person in the group. The notes are gone but the memory says it was lower—perhaps a third or a fourth of the "normal." Some interim reading—also not then noted—indicates that persons who become alcoholics must be more careful around electricity than others, as they are more easily electrocuted. There is no implication that persons with low electrical resistance tend toward or becom alcoholic victims, but there is an implication that bodily resistance to electr currents vary greatly and these variations have significant physical and physiological results. Medical research some 25 or 30 years ago, has established the fact that the human body is a generator as well as a conductor electricity. It is logical that generation and restivity would bring about a difference in the electrical field about the human body and its reaction with the field about water "veins."

Here, physiology and the physical sciences appear to have a common inte est. So far, nothing has gone beyond what Maby and Franklin have reported, so far as the writer knows. It would seem to be an interesting field for both the medical and the earth scientist to explore.

The question now arises—"so what?" Granted that the dowzer has defini powers, is he useful to the geologist or the hydrologist? Mr. Langbein himself answers that question. In the absence of completely detailed geologi cal and hydrologic subsurface knowledge, the dowzer certainly can be useful In the writer's opinion, the likelihood of any other knowledge supplanting hin appears remote. So far as the writer's knowledge of geology and hydrology permit, neither of the latter branches of science will produce the detailed knowledge available to the informed and experienced dowzer. It seems poss ble that an extension of the Maby and Franklin research could enhance the dowzer's powers, and mayhap even elevate him to a "professional" category

This would appear a subject of useful and profitable research. There certainly is no further basis for scoffing.

V. M. YEVDJEVICH,[1] M. ASCE.—Two very interesting problems are dis cussed by Mr. W. B. Langbein: a) contrast of the divining rods as an

1. Hydr. Engr., Foreign senior scientist, National Bureau of Standards, Washington, D. C.

logic decision-maker versus the modern basic-data programs; b) con-
of the increase of gathering basic-data versus the deficiencies in re-
n necessary to guide, analyze and interpret the basic data, and to obtain
ne general principles and the regional hydrologic characteristics.
s clear that modern hydrology does not support "forked stick" pro-
e which does not have a serious scientific background. But the hydrolo-
ust be aware of some facts which contribute to the persistence of the
r's activities, so that the hydrologist may overcome his seemingly ap-
disadvantages. Ground water hydrology, frequently very complex, im-
some uncertainty in decisions upon the geohydrologic specialist, while
wser's short cut way to decisions makes a spectacular impression. The
r is often a man well acquainted with local conditions, and makes de-
s by analogy, based on experiences. He is sometimes a kind of living
easy for a farmer to consult. He is available, and what is important,
. These are the reasons why the use of the divining rod is so persistent
ill continue to be so for relatively smaller activities until replaced by
engineering procedures, competitive from other points of view.
e collection of basic data in hydrology has been started and made under
general assumptions for the expected water resources development and
ol. The basic concepts laid for the collection of hydrologic data depend
he future water resources development. The first concepts were based
expected individual and mostly single purpose projects. Next step in
opment was the multiple purpose use of water resources. The future
robably ask the total and very integrated water resources development
tate of great demands for water control and use. The basic concepts
ning the collecting of hydrologic data today should anticipate the situation
er resources development 2 - 3 decades or more from now. This is
er reason to support Mr. Langbein's emphasis for a better scientific
tion in general collection and interpretation of basic-data.
en a demand for higher scientific efforts in hydrology is forwarded, the
sary problems connected with it have to be emphasized. The efficiency
research in hydrology is an important question. Surveying the hydro-
iterature of the countries with relatively abundant funds and large
er of people employed in research, the impression is that the results do
rrespond to the efforts. The quality of the results and the possibilities
e useful applications are rather limited in comparison with the quantity
lished papers and other literature. The greatest results for given funds
eir best application to the current needs and problems can be achieved
y timely and thorough preparations for expending research.
e timely solution of many problems can help the rapid development of
rch in hydrology, as for example: the systematic preparation of highly
ed engineers and scientists in hydrology; the development of good pro-
es in putting forward and in selecting the topics to be studied; the ex-
e of experiences, ideas and new methods; the type of work, in teams,
dual or both; etc.

QUEUING THEORY AND WATER STORAGE[a]

Discussion by B. W. Gould

W. GOULD.[1]—The author's paper shows that the techniques of mathe-
al statistics may be applied in the solution of the problem of determining
ed water storage, and also in estimating the probable behaviour of a
voir already constructed.
e service function:

$$D = b + kS$$

e interpreted in several different ways, each of which gives a different
.
avoid confusion, the writer has adopted the following notation:

y = Average rate of discharge in the interval between time x and time y.

y = Average rate of inflow in the interval between time x and time y.

= Instantaneous rate of discharge at time x.

= Volume of stored water at time x.

e unit of storage used by the writer is the same as that used by the
, being equal to unit flow for unit time.
e first interpretation of the service function to be considered is that the
rge during an interval of time is determined by the storage at the end
interval. This type of control would be difficult to achieve in practice,
considered because it gives the same solution as that obtained by the
.

$$D_{x,x+1} = b + kS_{x+1}$$

milarly

$$D_{x-1,x} = b + kS_x$$

sic equations obtained by the author may be verified by subtraction, and
ution of

$$I_{x,x+1} - D_{x,x+1} \quad \text{for} \quad (S_{x+1} - S_x$$

ther interpretation of the service function could be that the instantane-
scharge is dependent on the instantaneous value of storage, i.e.

---

c. Paper 1811, October, 1958, by W. B. Langbein.
r., Horsham Waterworks Trust, Vic., Aust., formerly lecturer in Civ.
., Univ. of New South Wales, Aust.

$$D_x = b + kS_x$$

and

$$D_{x+1} = b + kS_{x+1}.$$

By subtraction and substitution, this gives

$$D_{x+1} - D_x = k(I_{x,x+1} - D_{x,x+1})$$

If the time interval is sufficiently small, $D_{x,\,x+1}$ is approximately equal 1/2 $(D_x + D_{x+1})$. Substituting this value and rearranging gives:

$$D_{x+1} = \frac{2k}{2 + k} I_{x,x+1} + \frac{2 - k}{2 + k} D_x$$

instead of the author's

$$D_2 = \frac{k}{1 + k} I_2 + \frac{1}{1 + k} D_1$$

In yet another case, it may be taken that the discharge could be determi by the amount of water in storage at the beginning of an interval. This type of control would be suitable for irrigation of an area containing both semi-permanent (e.g. citrus or vines) and annual (e.g. market garden) developme where a certain minimum amount of water is required to maintain the sem permanent vegetation, whilst the area of annuals to be planted in any year be varied according to the amount of stored water available at the beginnin of the year. This gives the equations:

$$D_{x,x+1} = b + kS_x$$

and

$$D_{x-1,x} = b + kS_{x-1}$$

Subtracting, and substituting for $(S_x - S_{x-1})$ gives

$$D_{x,x+1} = kI_{x-1,x} + (1 - k)D_{x-1,x} .$$

An estimate of the discharge in the nth year, given by successive substi tution of "D" values, is:

$$D_{n-1,n} = k\ I_{n-2,n-1} + (1-k)I_{n-3,n-2} + (1-k)^2 I_{n-4,n-3}$$

$$\text{etc.} + (1-k)^{(r-1)} I_{n-r-1,n-r} + \text{etc.}$$

From this, the standard deviation of the estimate of the discharge can be calculated as.

$$\sigma_d = \sigma\sqrt{\frac{k}{2 - k}}$$

instead of

$$\sigma_d = \sigma\sqrt{\frac{k}{2 + k}}$$

ven by the author in Eq. (6).

comparison between the author's Eqs. (12), (13), and (14), and the corre-
ding equations derived by the writer for the case where the discharge
g a period is based on the storage available at the beginning of the peri-
s given in the following table:

n. o.	Discharge based on final storage.	Discharge based on initial storage.
2	$k = \dfrac{2(\bar{X} - b)^2}{(\bar{X}-m)^2 - (\bar{X}-b)^2}$	$k = \dfrac{2(\bar{X} - b)^2}{(\bar{X}-m)^2 + (\bar{X}-b)^2}$
3	$S = \dfrac{(\bar{X}-m)^2 - (\bar{X}-b)^2}{(\bar{X} - b)}$	$S = \dfrac{(\bar{X}-m)^2 + (\bar{X}-b)^2}{(\bar{X} - b)}$
4	$S = t\dfrac{\sigma^2 - \sigma_d^2}{\sigma_d}$	$S = t\dfrac{\sigma^2 + \sigma_d^2}{\sigma_d}$

is to be noted that where the discharge is determined by the amount of d water at the beginning of the period under consideration, the equations k = 1 when b = m. In this case, any flow in excess of the minimum flow y year may be considered as being stored for use in the immediately wing year, thus giving time to make plans for its utilization. Under such itions, the storage required to provide 100 per cent utilization would be ifference between the maximum probable inflow, and the minimum proba- atural inflow. This is verified by the equation which gives the required ge as $2t\sigma$.

s in the case considered by the author, the storage required increases ly as b approaches $\bar{x}$, and becomes infinite when b = $\bar{X}$.

e accuracy of the "probability routing" methods described by the author amples 2 and 3 in the paper, is limited only by the number of storage vals used, the number of successive approximations made, and the ever- ent errors of sampling. Electronic computation using standard rammes for the solution of linear simultaneous equations has been used e writer, who found that smaller intervals of storage volume could be and that the need for successive approximations was eliminated because olution was provided in a direct manner.

PROCEEDINGS PAPERS

hnical papers published in the past year are identified by number below. Technical-division p is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hy-HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors sions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with (January 1956) papers were published in Journals of the various Technical Divisions. To locate ne Journals, the symbols after the paper number are followed by a numeral designating the issue ular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) cates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division 8.

VOLUME 84 (1958)

580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), O2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)c, 1606(SA2), A2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)c, 1615(IR2)c, 1616 1617(SU1), 1618(PO2)c, 1619(EM2)c, 1620(CP1).

1(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), W3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)c, 1653(WW3)c, 1654(SM2), 1655(SM2), T3)c, 1657(SM2)c.

58(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)c, 1689(PO3)c, 1690(HY3)c, 1691(PL2)c.

92(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), A4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 1720(SU2), 1721(ST4)c, 1722(ST4), 1723(ST4), 1724(EM3)c.

1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

ER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), T5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), W4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 1785(WW4)c, 1786(SA5)c, 1787(ST5)c, 1788(IR3), 1789(WW4).

: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), W3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 1825(SM4), 1826(SM4), 1827(ST6)c, 1828(SM4)c, 1829(HW3)c, 1830(PO5)c, 1831(EM4)c, 1832(HY5)c

ER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), T7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), A6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)c, 1856(HY6)c, 1857(ST7)c, 1858

ER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 , 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)c, 1881(SM5)c, 1882(ST8)c, 1883(PP1)c, 1884(WW5)c, P2)c, 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), W1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), T1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)c, 1917(EM1)c, W1)c, 1919(HY1)c, 1920(PL1)c, 1921(SA1)c, 1922(ST1)c, 1923(EM1), 1924(HW1), 1925(HW1), 1926 1927(HW1), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

RY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), T2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), Y2)c, 1951(SM1)c, 1952(ST2)c, 1953(PO1)c, 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958 1959(CO1).

1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977 , 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)c, R1)c, 1987(WW1)c, 1988(ST3)c, 1989(HY3)c.

1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 c, 2008(EM2)c, 2009(ST4)c, 2010(SM2)c, 2011(SM2)c, 2012(HY4)c, 2013(PO2)c.

sion of several papers, grouped by divisions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1959

PRESIDENT

FRANCIS S. FRIEL

VICE-PRESIDENTS

Term expires October, 1959:
WALDO G. BOWMAN
SAMUEL B. MORRIS

Term expires October, 1960:
PAUL L. HOLLAND
LLOYD D. KNAPP

DIRECTORS

Term expires October, 1959:
CLINTON D. HANOVER, Jr.
E. LELAND DURKEE
HOWARD F. PECKWORTH
FINLEY B. LAVERTY
WILLIAM J. HEDLEY
RANDLE B. ALEXANDER

Term expires October, 1960:
PHILIP C. RUTLEDGE
WESTON S. EVANS
TILTON E. SHELBURNE
CRAIG P. HAZELET
DONALD H. MATTERN
JOHN E. RINNE

Term expires October, 19
THOMAS J. FRATAR
EARL F. O'BRIEN
DANIEL B. VENTRES
CHARLES W. BRITZIU
WAYNE G. O'HARRA
FRED H. RHODES, JR
N. T. VEATCH

PAST PRESIDENTS

Members of the Board

MASON G. LOCKWOOD

LOUIS R. HOW

EXECUTIVE SECRETARY

WILLIAM H. WISELY

TREASURER

CHARLES E. TROUT

ASSISTANT SECRETARY

E. LAWRENCE CHANDLER

ASSISTANT TREASURER

ENOCH R. NEEDLES

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

PAUL A. PARISI
Editor of Technical Publications

MARVIN SCHECHTER
Assistant Editor of Technical Publicati

COMMITTEE ON PUBLICATIONS

HOWARD F. PECKWORTH, *Chairman*

PHILIP C. RUTLEDGE, *Vice-Chairman*

E. LELAND DURKEE

CHARLES W. BRITZIUS

TILTON E. SHELBURNE

FRED H. RHODES, JR.